CDM

City of Benicia

Water Reuse Project
Phase One

Conceptual Design Report

CDM

One Walnut Creek Center
100 Pringle Avenue, Suite 300
Walnut Creek, CA 94596
tel: 925 933-2900
fax: 925 933-4174

October 23, 2006

Ms. Chris Tomasik
Assistant Director of Public Works
City of Benicia
250 East "L" Street
Benicia, CA 94510

Subject: Water Reuse Project – Conceptual Design Report

Dear Chris:

CDM is pleased to submit the Conceptual Design Report for the Benicia Water Reuse Project. The report is a summary of the information presented in the project's five technical memoranda that have been submitted over the last two years, updated and revised to incorporate new project cost estimates, and to include the input received the City and the PURE committee.

This report would not have been possible without the valuable input and guidance from you and your staff and the PURE Committee. If you have any questions or need additional information, please do not hesitate to call us.

Very truly yours,

T. Gerald Cole, P.E.
Senior Project Manager
Camp Dresser & McKee Inc.

Paul F. Meyerhofer, P.E.
Senior Vice President
Camp Dresser & McKee Inc.

Contents

Contents

Contents

Tables

Figures

Page

Executive
Summary

Executive

Executive Summary

The City of Benicia and the nearby Valero Refinery have entered into a partnership to develop a project that will supply recycled water for use as cooling tower make-up water. The project is being developed to deliver up to 2 mgd of high purity recycled water to the refinery, which is approximately three miles north of the City's WWTP. The overall project objectives as established by the City and Valero are as follows:

- Meet water quality and quantity requirements for the cooling towers
- Meet discharge requirements for disposal of demineralized reject stream
- Comply with State Title 22 requirements for recycled water for cooling towers

Table ES-1 presents a listing of secondary effluent constituents of concern and the limits required for the recycled water to meet the water quality criteria.

Table ES-1
Comparison of Key Secondary Effluent Quality Parameters and Recycled Water Quality Limits

Parameter	*Units*	*Benicia Effluent Water Quality*	*Cooling Water Quality Limits*
ammonia	mg/L	30	<0.2
bicarbonate	mg/L	190	104
chloride	mg/L	120	20
phosphate	mg/L	2	3
silica	mg/L	22	17
hardness	mg/L	130	<200
TDS	mg/L	650	250

Ammonia Removal

In order to provide assurance that the best ammonia removal technology was selected, several treatment technologies were evaluated. Three alternatives involved modifications to the City's existing WWTP. They would require that the entire secondary treatment system be included in the process development, along with accommodations for wet weather operations. Three other alternatives analyzed were basically stand alone systems, which were sized solely to meet the flow demands of the Water Reuse Project.

Based on the evaluation of the alternatives, it is recommended that stand-alone nitrifying trickling filters be selected as the nitrification system to be used in the overall process system for the Benicia Water Reuse Project.

Partial Demineralization

Computer simulation models of alternative partial demineralization treatment processes were run to determine the most cost-effective system that could process the City's effluent to meet the cooling water quality objectives. Technologies investigated in various combinations, included: granular media filtration, microfiltration (MF), nanofiltration (NF), reverse osmosis (RO) and electrodialysis reversal (EDR).

MF followed by RO was determined by computer simulations to meet all the requirements except for ammonia. Reducing the ammonia from about 0.3 mg/L after RO to less than 0.2 mg/L will be achieved by breakpoint chlorination after disinfection.

Using the MF/RO processes described above, the recycled water quality is projected to meet the water quality objectives, as shown in Table ES-2

Table ES-2
Comparison of Key Secondary Effluent Quality Parameters and Recycled Water Quality Limits and Projected Recycled Water Quality

Parameter	Units	Benicia Secondary Effluent Water Quality	Cooling Water Quality Limits	Projected Recycled Water Quality[(a)]
ammonia	mg/L	30	<0.2	<0.2
bicarbonate	mg/L	190	104	37
chloride	mg/L	120	20	<20
phosphate	mg/L	2	3	0.5
silica	mg/L	22	17	4
hardness	mg/L	130	<200	23
TDS	mg/L	650	250	120

[(a)]Based on 15 % blend around the RO system and breakpoint chlorination

Disinfection

Alternative disinfection systems evaluated for the Benicia Reuse Project included chlorination using sodium hypochlorite and ultraviolet light disinfection. Recycled water from the proposed Water Reuse Treatment System must meet disinfection requirements for tertiary recycled water, proposed for use as cooling water supply, as contained in Title 22, Division 4, Chapter 3 of the California Code of Regulations. Two disinfection system alternatives were developed and evaluated, namely low-pressure, high intensity UV and chlorination using sodium hypochlorite. An economic analysis indicated that chlorination and UV disinfection are approximately equal in cost. Moreover, other qualitative factors, in particular water quality impacts, site impacts and ease of process control, favor UV over chlorination.

Recycled Water Conveyance System

The conveyance system will consist of a pump station at the City of Benicia WWTP, a pipeline approximately 14,000 feet in length and a "break tank" storage facility at the Refinery. Beginning at the WWTP the pipeline will travel from a new, high-lift recycled water pump station (RWPS) to the Valero "off site" dock line right-of-way in the vicinity of East 7th Street and "H" Street. The pipeline will follow the abandoned

Valero dock lines northerly for about 9,000 feet to the Refinery property line. Within the Refinery the pipeline will follow Avenue "E" South, then up a vertical rise (known as a "waterfall") to Avenue "F" to the cooling towers.

Rehabilitating the existing dock lines was compared to constructing new piping. It was determined that it was more cost-effective and reliable to install a new, 14-inch pipeline, rather than rehab portions of the existing dock lines. The recycled water pump station will consist of three (2 duty/1 standby), vertical turbine, variable speed pumps mounted over a clearwell.

A flow diagram of the proposed Benicia Water Reuse Plant and Conveyance System is shown in Figure ES-1.

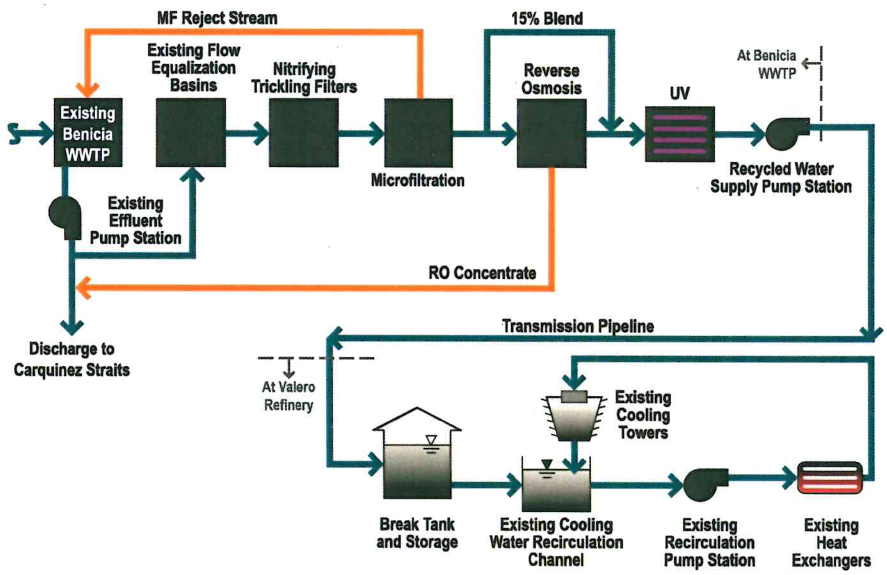

Figure ES-1
Benicia Water Reuse Plant and Conveyance System
Flow Diagram

Regulatory Compliance And Pilot Testing

The reject (or concentrate) stream from the RO facility will be blended with the remaining Benicia WWTP flow and discharged to the Carquinez strait. Constituents in the RO concentrate stream will be concentrated up to fives times higher than levels in the secondary effluent that will feed the MF/RO treatment system.

Initial planning level estimates indicate that up to 0.3 mgd of concentrate could be produced from the full-scale RO facility when operating at maximum capacity. That flow would be blended and discharged with the remaining approximately 0.4 mgd (minimum) of secondary effluent (i.e. a 43% blend).

Pilot-scale tests and laboratory analyses were performed to investigate the feasibility of the blended discharge (RO concentrate and Benicia WWTP effluent) meeting current NPDES discharge requirements and to characterize the following:

- Conventional water quality parameters (BOD, TSS, pH, etc.).
- Trace metals and other priority pollutants
- Acute and chronic toxicity.

The results of the testing and analyses indicate that the blended discharge will meet regulatory requirements.

Estimated Costs

Construction costs were estimated for the water reuse plant and conveyance system. The capital cost of a project includes both the construction cost plus all "soft costs" that are required to implement the project. These soft costs include: engineering, construction management, administration, environmental compliance, acquisition of permits and financing costs. The assumptions used in developing capital cost estimates are:

- Estimates include 25% for engineering design and construction management, 25% for contingencies, and $1 million for the preliminary engineering, water quality testing, and environmental planning costs that will be completed prior to the start of engineering design.
- The project will be bid in May, 2008.
- The contractor will price the project to the mid-point of construction (May 2009).
- Construction cost escalation between October 2006 and May 2009 will range between 6% and 12% annually.

Table ES-3 presents a summary of the estimated capital costs for water reuse projects with production capacities of 2.0, 1.5 and 1.0 mgd, respectively.

Table ES-3 Summary of Estimated Capital Cost			
	2.0 mgd Water Reuse Project	1.5 mgd Water Reuse Project	1.0 mgd Water Reuse Project
Component	**Cost ($ millions)**		**Cost ($ millions)**
Construction	$18.68	$15.84	$12.31
Engineering and CM at 25%	$4.67	$3.96	$3.08
Subtotal	$23.35	$19.80	$15.39
Contingency at 25%	$5.84	$4.95	$3.85
Costs for preliminary engineering, water quality testing, and environmental planning	$1.00	$1.00	$1.00
Total Cost based on Oct. 2006	$30.19	$25.75	$20.24
Total Capital Cost, assuming 6% annual inflation to mid-point of construction in May 2009 (2.5 yrs)	**$34.92**	**$29.80**	**$23.40**
Total Capital Cost, assuming 12% annual inflation to mid-point of construction in May 2009 (2.5 yrs)	**$40.14**	**$34.25**	**$26.90**

The O&M costs of the project include power, labor, chemicals, and replacement of consumables (e.g., membranes, UV lamps, etc). Labor estimates were based on experience with other operations at plants, available guidelines and discussions with existing Benicia Plant operations staff. The replacement costs for major consumables were based on manufacturers' recommendations and experience with other projects.

A summary of estimated O&M costs is presented in **Table ES-4.**

Table ES-4 Estimated O&M Costs			
Item	***2 mgd Water Reuse Project***	***1.5 mgd Water Reuse Project***	***1.0 mgd Water Reuse Project***
Chemicals	$270,800	$203,200	$135,500
Power	$400,400	$300,400	$200,400
Consumables	$162,500	$121,800	$81,400
ER&R	$99,500	$89,240	$79,140
Labor	$239,500	$239,500	$239,500
E and I&C Maint.	$50,000	$50,000	$50,000
Total	**$1,222,700**	**$1,004,140**	**$785,940**

1

Section One

Section 1
Introduction

1.1 Project Background and Objectives

The City of Benicia and the nearby Valero Refinery have entered into a partnership to develop a project that will supply recycled water for use as cooling tower make-up water. The recycled water will off-set a commensurate amount of raw water, thus increasing the reliability of the City's potable supply.

The City of Benicia is located in the southwest corner of Solano County on the San Francisco Bay. The City owns and operates a secondary treatment plant with a design capacity of approximately 4 mgd. The plant provides secondary treatment by an activated sludge process and discharges its effluent to the Carquinez Strait of the San Francisco Bay. Current average daily, dry weather flow during summer months is approximately 2.7 mgd. Effluent quality discharge requirements (monthly average) are 30 mg/L BOD and 30 mg/L suspended solids. Toxicity limits (chronic and acute) and toxic substances, particularly heavy metals, are also regulated by the City's NPDES permit.

The project is being developed to deliver up to 2 mgd of high purity recycled water to the refinery, which is approximately three miles north of the City's WWTP. **Figure 1-1** presents a project location map.

The overall project objectives as established by the City and Valero are as follows:

- Meet water quality and quantity requirements for the cooling towers.
- Meet discharge requirements for disposal of demineralized reject stream.
- Comply with State Title 22 requirements for recycled water for cooling towers.

1.2 Project Authorization

On July 7, 2004, in accordance with Task Order No. 1 to the Consultant Agreement between the City and CDM, the City of Benicia authorized CDM to provide Phase One Engineering Services for the development of the proposed Water Reuse Project. The Scope of Work includes reviewing existing background documents, conducting small scale pilot testing, and developing conceptual and preliminary designs for the proposed project.

1.3 Technical Memoranda

In the development of the conceptual design, CDM prepared five Technical Memoranda each for various components of the Project. The technical memoranda were prepared in draft form and were submitted to the City and its steering

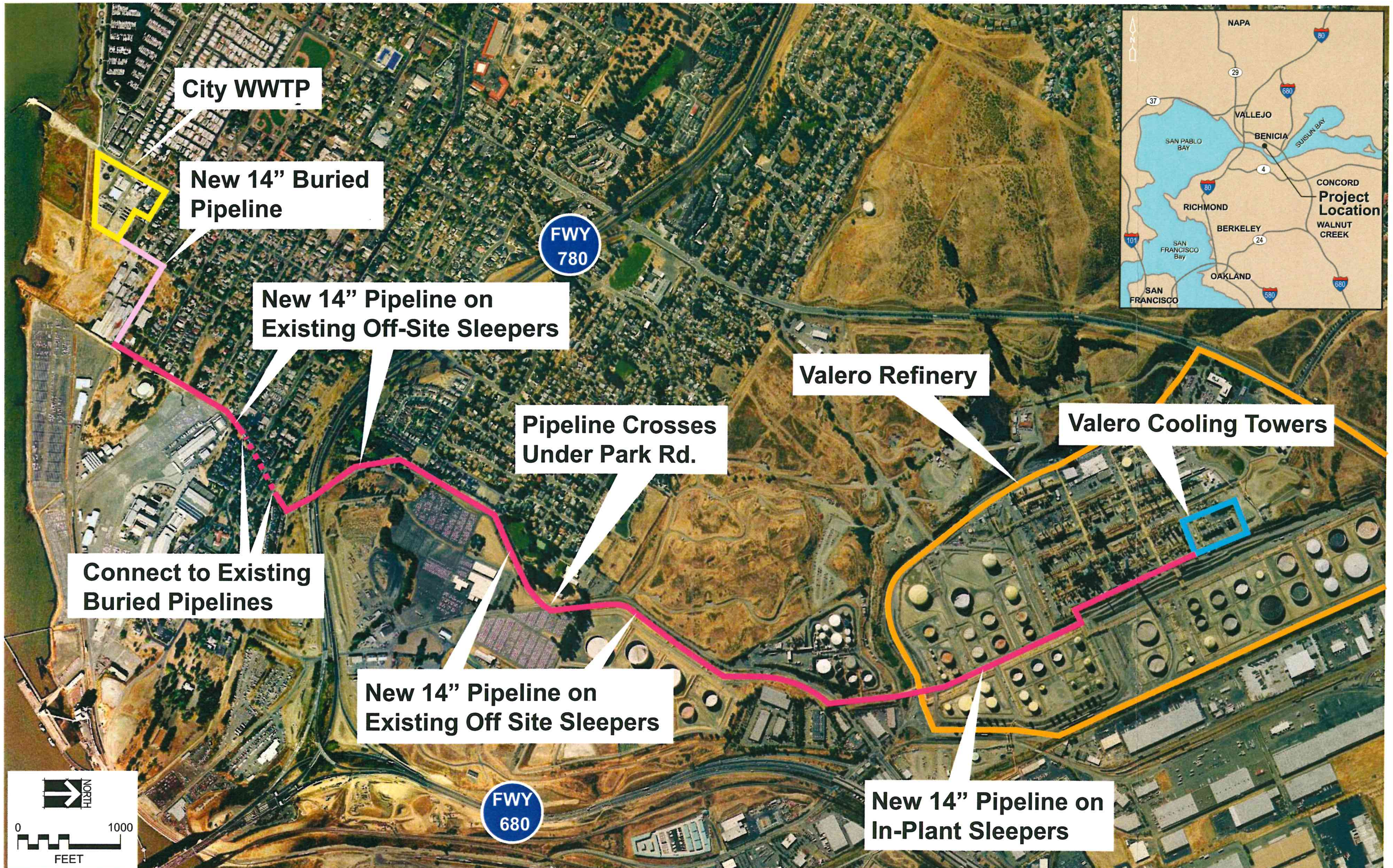

Figure 1-1
City of Benicia Water Reuse Project Overall Site and Alignment Map

committee (described below) for review, comment and approval of CDM's recommendations. The technical memoranda produced in the development of the project concept are as follow:

- TM 1 – Evaluation of Alternative Reuse Treatment Systems and Ammonia Removal Options (Sept. 7, 2004)
- Supplement to TM 1 – Biological Nitrification Alternatives (Nov. 30, 2005)
- TM 2 – Evaluation of Alternative Disinfection Processes (Nov. 4, 2004)
- TM 3 – Recycled Water Conveyance System (Nov. 9, 2004)
- TM 4 – Analysis of Facilities Siting Alternatives (Feb. 2, 2005)

This conceptual design report is a compilation and summary of these technical memoranda. The technical memoranda are found in the Appendix.

1.4 Project Team

The Project is being developed under the direction of Chris Tomasik, Assistant Director of Public Works for the City.

In addition to CDM, the City has retained EOA, Inc to provide consultation and direction for permitting and regulatory compliance issues. Pacific Eco-Risk Laboratories performed toxicity studies relating to the disposal of the reverse osmosis (RO) concentrate (or brine reject stream).

To ensure that the project meets the requirements of the California Environmental Quality Act (CEQA), the City has retained ESA to perform environmental assessment of the project and to develop the appropriate CEQA compliance document.

1.5 Acknowledgements

CDM wishes to acknowledge the valuable guidance and expert advice received from the City, and in particular, Chris Tomasik, John Bailey (retired WWTP Superintendent), Jerry Gall, WWTP Superintendent, and Jeff Gregory, WWTP Supervisor.

The City has an ad hoc steering committee for the Project, known as PURE (People Using Resources Efficiently). CDM is also very appreciative of the guidance, insight and direction provided from the committee as a whole and individually. The PURE committee is comprised of Benicia residents appointed by the City Council. The members are: Robert Craft, Chair; Donald Basso; Dennis Lund; Brad MacLane; and Elizabeth Patterson, Council member. The Valero Refinery representative to PURE is Guy Young.

Lastly, CDM appreciates the partnering relationship exhibited by the other consultants retained by the City, in particular Tom Hall of EOA, with whom CDM worked closely in the scoping and conduct of the pilot testing and data analyses.

List of Acronyms

AB	aeration basin
ADWF	Average Dry Weather Flow
AF	acre-feet
AFY	acre-feet per year
AS	Activated Sludge
AWWA	American Water Works Association
BAAQMD	Bay Area Air Quality Management District
BAF	biological aerated filter
BFP	belt filter press
BNR	Biological Nutrient (Nitrogen) Removal
BOD	biochemical oxygen demand
BOD_5	5-day Biochemical Oxygen Demand
BTU	British Thermal Unit
CAA	Clean Air Act
$CaCo_3$	Calcium Carbonate
CCR	California Code of Regulations
CDM	Camp Dresser & McKee Inc.
CEPT	chemically enhanced primary treatment
cf	cubic foot
CFR	Code of Federal Regulations
CIP	clean-in-place
COD	Chemical oxygen demand
COE	U.S. Army Corps of Engineers
CPI	Consumer Price Index
CT	Product of chlorine dosage and contact time
CWA	Clean Water Act
DAF	Dissolved Air Flotation Thickener
DG	Digester Gas
DL	Dockline
DO	dissolved oxygen
DOHS	State of California Department of Health Services
EDR	Electrodialysis Reversal
EHRC	enhanced high rate clarification
$ENRCCL_{SF}$	Engineering News Record Construction Cost Index of San Francisco Area
EPA	United States Environmental Protection Agency
FOTE	field oxygen transfer efficiency
FY	Fiscal year

gpd	gallons per day
gpm	gallons per minute
HDPE	High Density Polyethylene
HRT	Hydraulic Residence Time
icfm	inlet cubic feet per minute
IDI	Infilco Degremont Incorporated
IFAS	integrated fixed film activated sludge
kV	KiloVolt (1000 Volts)
kW	KiloWatt (1000 Watts)
kWhr	kilowatt hour
L	liter
MBR	membrane bioreactor
MF	microfiltration
mg	milligram
mgal	million gallons
mg/L	milligram per liter
mgd	million gallons per day
mL	milliliter
mL/L – hr	Milliliter per liter per hour
MLSS	Mixed Liquor Suspended Solids
mW	MegaWatt (1,000,000 Watts)
NAS	nitrifying activated sludge
NBA	North Bay Aqueduct
NF	nanofiltration
NPDES	National Pollutant Discharge Elimination System
NTF	nitrifying trickling filters
NTU	Nephelometric Turbidity Unit
O&M	Operation and Maintenance
OH	overhead
OSHA	Occupational Safety and Health Administration
PE	primary effluent
PLC	programmable logic controller
POTW	Publicly Owned Treatment Works
ppd	pounds per day
PS	pump station
PSM	Process Safety Management
PVC	polyvinyl chloride
PW	present worth
PWWF	peak wet weather flow
RAS	return activated sludge
RBC	rotating biological contactors
RO	reverse osmosis
RWQCB	Regional Water Quality Control Board – San Francisco Bay Region
RWSPS	recycled water supply pump station
SC	secondary clarifier

SCADA	Supervisory Control and Data Acquisition System
sf	square feet
SPW	State Project Water
SRT	solids (biomass) retention time
Sta	Station
SWRCB	State Water Resources Control Board
TDS	Total Dissolved Solids
Therm	100,000 BTUs, equivalent to 100 cubic feet of natural gas
TIN	Total Inorganic Nitrogen (total of ammonia-nitrogen, nitrite-nitrogen, and nitrate-nitrogen)
Title 22	California Code of Regulations, Title 22 (Water Recycling Criteria)
TKN	total Kjeldahl nitrogen
TOC	Total Organic Carbon
TSFF	Tertiary Submerged Fixed Film (nitrification)
TSS	Total Suspended Solids
μg/L	micro-grams per liter
USDA	U.S. Department of Agriculture
UV	ultraviolet light
UVT	UV transmittance
VOC	Volatile Organic Carbon
WRTP	Water Reuse Treatment Plant
WWTP	Wastewater Treatment Plant

2 Section Two

Section 2
Basic Criteria for Project Development

2.1 Introduction

Recycled water from municipal wastewater treatment plants is used in several locations for industrial cooling applications. In California, the West Basin Municipal Water District (West Basin) supplies high quality recycled water to Chevron's El Segundo Refinery and to Exxon/Mobil's Torrance Refinery. Chevron is developing a similar project at its Richmond, California Refinery. The Cooling Water Institute lists several other projects where recycled water is being used.

The major water quality constituents of concern when considering the application of recycled water for industrial cooling include ammonia, chloride, silica, total hardness, total dissolved salts (TDS) and others. The concerns generally focus on corrosion and/or plating out of minerals within heat exchangers and cooling towers.

Concerning ammonia, many municipal biological wastewater treatment plants do not nitrify (i.e., convert ammonia to nitrate), and typical ammonia concentrations in the secondary effluent from these plants range between 20 and 35 mg/L. However, very low ammonia levels (less than 0.2 mg/L) must be maintained for cooling water. Thus, ammonia removal steps must be implemented in recycled water plants. Alternatives include modifying the entire secondary biological process, providing a stand alone biological system, or implementing an ion exchange treatment process. As described in Technical Memorandum No. 1, early in this project it was determined that biological ammonia removal would be the most cost effective and practical. Therefore, biological ammonia removal (conversion) was investigated for the Benicia Water Reuse Project. These evaluations are described in Section 3 below and in more detail in the supplement to TM 1.

Owing to strict limits for the other mineral constituents noted above, some form of demineralization is necessary to meet cooling water objectives. Projects such as those implemented by West Basin employ reverse osmosis (RO) to reduce the chloride, TDS, and other minerals to the levels required by refinery cooling systems. As is described in Section 3 and TM 1, various membrane systems were investigated to meet the water quality requirements for the Benicia Water Reuse Project.

In the application of secondary effluent to RO membranes, pretreatment using micro-filtration (MF) or ultra-filtration is typically used to prevent RO membrane fouling and to extend the lives of the membranes. At West Basin and other projects, MF is used as the pretreatment to the RO process.

Based on CDM's experience with the West Basin project and other specific evaluations for this project, it was determined that some form of biological nitrification system followed by MF and RO would be the general overall process used to meet the water quality requirements.

As described in Section 3 and TM 2, various disinfection methods after the MF/RO systems were also evaluated and UV disinfection was selected.

2.2 Recycled Water Quality Objectives

Strict water quality objectives have been established relating to ammonia, silica, chloride and TDS. The bases for setting strict limits for these constituents are as follows:

- Corrosion of admiralty metals, e.g., copper-zinc alloys from chloride and ammonia.
- Plating out of deposits, e.g., $CaCO_3$.
- Build-up of slimes in cooling towers.
- TDS build up affects the number of cycles of concentration, which directly affects operating costs.

Table 2-1 presents a listing of secondary effluent constituents of concern and the limits required for the recycled water to meet the water quality criteria.

Table 2-1
Comparison of Key Secondary Effluent Quality Parameters and Recycled Water Quality Limits

Parameter	*Units*	*Benicia Effluent Water Quality*	*Cooling Water Quality Limits*
ammonia	mg/L	30	<0.2
bicarbonate	mg/L	190	104
chloride	mg/L	120	20
phosphate	mg/L	2	3
silica	mg/L	22	17
hardness	mg/L	130	<200
TDS	mg/L	650	250

2.3 Project Output Capacity and Flow Equalization

As is typical with most municipal wastewater systems, flow rates both into and out of wastewater treatment plants have considerable variation throughout the day as well as seasonally. Benicia is no exception. During dry weather periods, flow rates vary from about 1 mgd to peaks of nearly 4 mgd. During wet weather periods, flow rates can range from about 2 mgd to over 20 mgd. The Water Reuse Project needs to take these flow variations into account since one of the overall project objectives is to supply recycled water at a more or less constant rate of 2 mgd throughout the day.

To meet the design output capacity of the project, the input secondary effluent flow to the recycled water treatment system needs to be higher than 2.0 mgd to account for the reject (waste) flows from both the micro-filtration (MF) and the reverse osmosis (RO) processes. The MF process will reject about 10% of the input and the RO will reject approximately 15% of its input flow. The MF reject will be recycled to the headworks of the plant for reprocessing; the RO reject will be sent to the outfall for disposal.

Based on the preliminary flow balance performed by CDM, the input flow to the biological ammonia removal system will need to be approximately 2.55 mgd to account for the reject flows. Hence, a constant flow of secondary effluent must be made available at the rate of 2.55 mgd.

The MF and RO processes perform best when operated at nearly a constant flow rate. It is more cost-effective to equalize secondary effluent supply to the water reuse treatment system, than to equalize the product recycled water. Operating the water reuse treatment system at a constant flow rate also provides for stable operating conditions with less variation in process performance.

The secondary effluent will be equalized using a portion of the existing multi-purpose basins (MPBs) at the WWTP. These basins are generally used to equalize high, wet-weather flows to maintain the plant's performance during high flow periods. They are also used during dry periods to store wastewater when a process unit is taken out of service.

Secondary effluent flow will be diverted into the MPBs and will be withdrawn at a constant rate and sent to the biological ammonia removal system which is described in Section 3.

Figure 2-1 graphically shows the variation in plant flow rate during dry weather periods and the estimated amount of equalization storage required to deliver approximately 2.55 mgd on a continuous basis. Approximately 400,000 gallons of storage is required, which is approximately the volume of MPBs Nos. 3, 4 & 5. During high wet weather flow periods, equalizing flow will not be necessary.

2.4 Location of Project Facilities

Three siting alternatives were developed, based on the location of major process treatment components, as follows:

- Alternative No. 1 – All treatment facilities (MF/RO/UV) at Benicia WWTP
- Alternative No. 2 – MF and UV at the Benicia WWTP and the RO system at Valero
- Alternative No. 3 – MF at the Benicia WWTP and the RO and UV systems at Valero

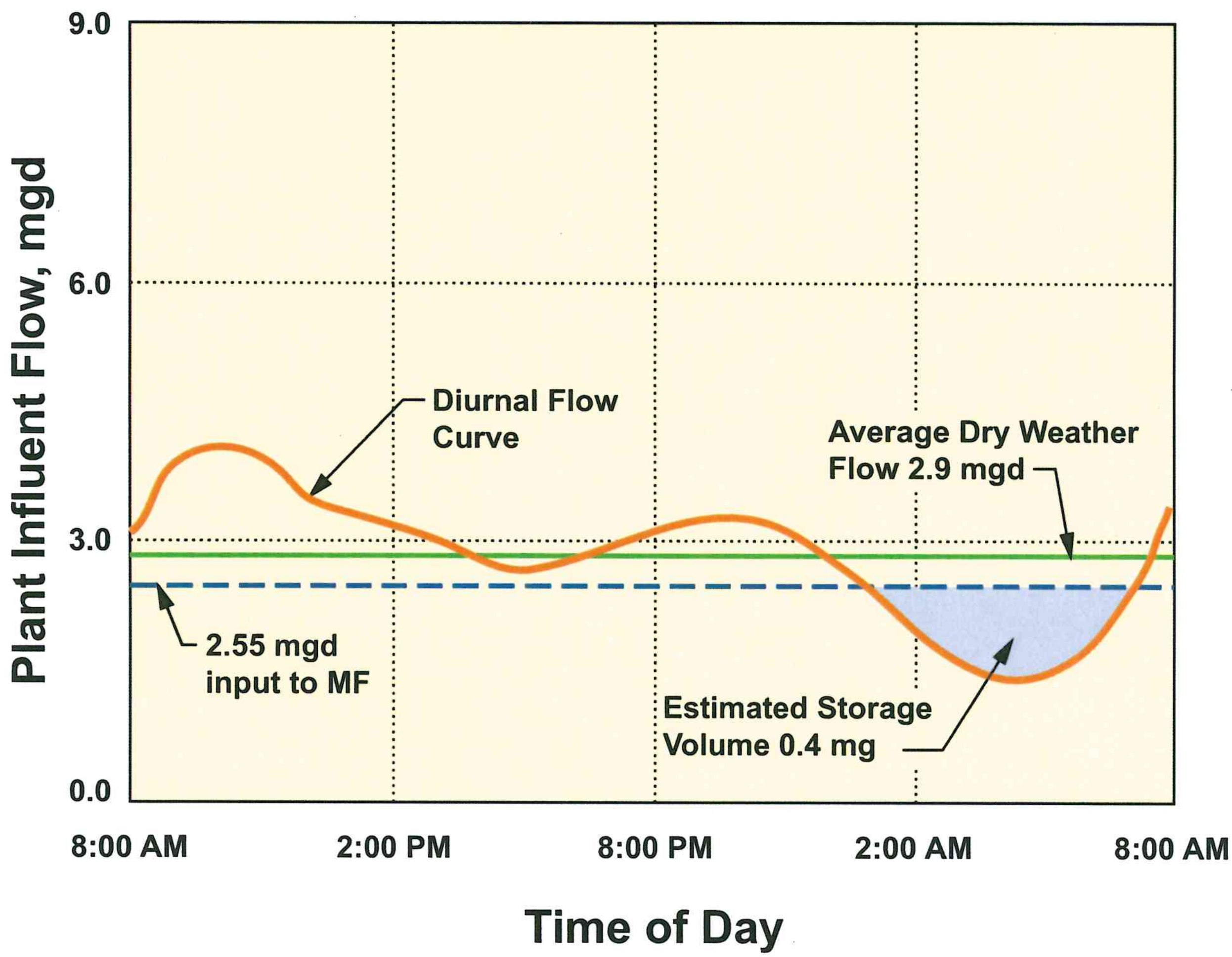

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Figure 2-1
Diurnal Storage Volume Required for 2.55 mgd Flow to MF

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Flow design criteria were established for the three alternatives, depending on where the facilities would be located. A present worth analysis was performed and it was determined that based on economics, lack of adequate available space at the refinery, and potential regulatory issues associated with RO concentrate disposal at Valero, all project treatment facilities would be located at the City's WWTP.

2.5 System and Process Reliability Criteria

During development of the conceptual design, project reliability issues were discussed with the City, Valero, and the PURE Committee. It was agreed that providing 100% project reliability (24/7/365) would be too costly. Hence, interruption in the delivery of recycled water could be tolerated by Valero. Some product water storage will be provided at the Refinery for limited power outage durations (volume to be determined). The City agreed that fresh water backup would remain available.

Based on the above decisions, the following criteria were developed:

- No Standby Power will be provided.
- All main line pump systems will have a standby pump.
- There will be two nitrifying trickling filters.
- MF system will be designed with multiple skids (minimum of 3).
- RO system will not be designed with a redundant skid, since there are no mechanical components associated with the system that are prone to fail.
- UV will be designed in compliance with the redundancy requirement of the Department of Health Services.

2.6 Other Planning Criteria

Meet City noise ordinance and minimize noise from project equipment.

3 Section Three

Section 3
Process Development and Selection

3.1 Description of City's Existing WWTP

To understand project development and process selection, it is necessary to understand the City's existing WWTP. The plant has two separate biological treatment systems. The conventional activated sludge system, which was added in the late 1990's, has a capacity of 4 mgd, but can handle up to 8 mgd peak flows during wet weather periods. It also has an RBC system that was constructed in the 1970's. The RBC system is used during wet weather, when peak flows exceed 12 mgd. Flows above 12 mgd are stored in the multi-purpose basins for equalization. Primary and waste activated sludges are gravity thickened and anaerobically digested. Digested sludge is dewatered on a belt press and the cake is hauled to a landfill. **Figure 3-1** presents a process block diagram of the liquid stream of the plant.

3.2 Development and Evaluation of Biological Nitrification Treatment Alternatives

Eleven biological treatment technologies that would potentially provide full-time nitrification were identified and screened. Six biological nitrification technologies were selected for further analysis. Three alternatives involve extensive modifications to the City's existing WWTP. They require that the entire secondary treatment system be included in the process development, along with accommodations for wet weather operations. Three other alternatives are basically stand alone systems, which can be sized solely to meet the flow demands of the Water Reuse Project. The six alternatives are described in **Table 3-1.**

Conceptual designs were prepared for each alternative and analyzed for performance, reliability and cost-effectiveness. The results of this analysis are described in the following paragraphs.

Table 3-1
Biological Nitrification Alternatives for Ammonia Removal

Alternative	***Description***
1	Expand existing activated sludge system – use 2 existing aeration basins (AB's) add a 3rd secondary clarifier (SC), 3rd return activated sludge (RAS) pump and 3 process air blowers. *Nitrifying Activated Sludge (2 AB's & 3 SC's)*
2	Expand existing activated sludge system – add 3rd AB, 3rd SC, 3rd RAS Pump and 3 blowers. *Nitrifying Activated Sludge (3 AB's & 3 SC's)*
3	Convert primaries to chemically enhanced primary treatment (CEPT) - add a 3rd secondary clarifier, 3rd RAS pump, 3 process blowers and chemical feeding system. *Nitrifying Activated Sludge & CEPT*
4	Add stand-alone tertiary nitrifying biological aerated filters. *Nitrifying BAF's*
5	Add stand-alone tertiary submerged, fixed-film nitrification system. *TSFF Nitrification*
6	Add stand-alone tertiary nitrifying trickling filters. *NTF's*

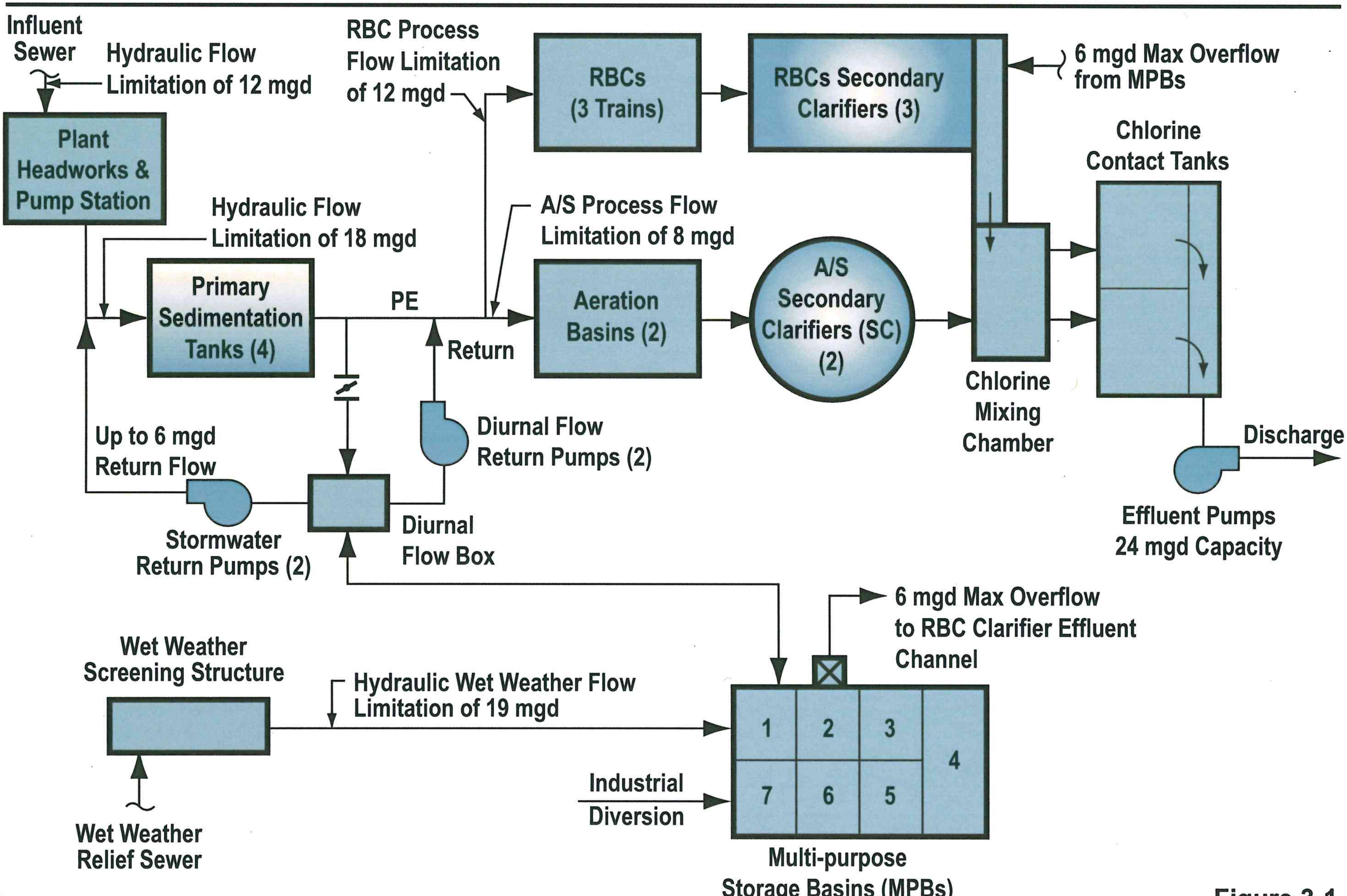

Figure 3-1
City of Benicia WWTP Existing Process
Schematic Diagram

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W:\REPORTS\Benicia\Conceptual Design Report\Graphics\City of Benicia WWTP Existing Process (Fig3-1).ai 10/20/06

3.2.1 Overview Nitrifying Activated Sludge Alternatives (Alternative Nos. 1, 2 & 3)

Of the three NAS alternatives, Alternative No. 2 provides the highest degree of reliability because nitrification can be maintained during wet weather flows with either one aeration basin (AB) or one secondary clarifier (SC) out of service. Alternative No. 3 provides less reliability than Alternative No. 2 because nitrification will likely be lost when one AB is removed from service; however, the activated sludge process can still pass the required wet weather flow with one SC out of service. Of the three full plant nitrifying activated sludge processes, Alternative No. 1 provides the lowest level of reliability because loss of an AB will stop nitrification and loss of a SC will prevent the SCs from passing the required wet weather flow. **Table 3-2** presents a summary of the flow rates that each of these three alternatives can handle and still reliably meet the secondary effluent ammonia limit of 2 mg/L.

Table 3-2
Summary of Reliable Flow Limitations for Nitrifying Activated Sludge Alternatives

Alternative	*Average Day Max Month, mgd*	*Peak Hourly Flow, mgd*
Estimated Flow at Build Out	4.5	8
Current Flow	3.7	8
Alt No. 1 – NAS with 2 AB's & 3 SC's	2.0 to 3.2	5.4 to 8.4
Alt No. 2 – NAS with 3 AB's & 3 SC's	3.2 to 4.0	8.3 to 10.8
Alt No. 3 – NAS & CEPT with 2 AB's & 3 SC's	2.4 to 3.6	6.4 to 9.9

3.2.2 Overview of Stand-Alone Biological Nitrification Systems

3.2.2.1 Alternative No. 4 Biological Aerated Filters

Biological aerated filters (BAF's) are a type of attached growth biological treatment process that is used for tertiary nitrification. Nitrifying bacteria grow on the surface of the media and convert the ammonia to nitrate. BAF's have characteristics of both activated sludge systems and trickling filters. They function similar to a water filter in that they must be backwashed periodically. Hence, there is backwash wastewater that must be recycled back to the main plant head works. The system has backwash pumps, process air blowers and backwash air blowers. BAF's are approximately 25 ft in height.

3.2.2.2 Alternative No. 5 Tertiary Submerged Fixed-Film Reactor Systems

Tertiary submerged fixed-film (TSFF) reactor systems are composed of a reaction vessel in which nitrifying bacteria grow on either fixed or moving-bed media. Air is diffused into the water-media culture much like a typical activated sludge (AS) aeration basin. Fixed media consist of either ropes that are attached to frames, or plastic crates, similar to those used in packed bio-towers. Moving-bed media are made of either sponges or small plastic elements. Since maintenance of the fixed media has presented challenges at some installations, only plastic media of the moving-bed type were considered. TSFF systems have low profiles, are similar to aeration basins and would project about five feet above grade.

3.2.2.3 Alternative No. 6 Tertiary Nitrifying Trickling Filters

Nitrifying trickling filters (NTF) are attached growth biological treatment processes that allow the nitrifying bacteria to grow on the surface of solid media, as the wastewater flows over the media. This is opposite of the suspended growth processes (i.e., NAS, as in Alternative Nos. 1, 2 & 3 and TSFF systems, as in Alternative No. 5) where the bacteria are suspended in the wastewater. The NTF's units for Benicia would be approximately 42-ft in diameter and 15-ft high.

3.2.3 Estimated Construction Costs of Biological Nitrification Alternatives

Conceptual designs were developed and construction cost estimates were prepared for each of the six alternatives. For the three stand-alone alternatives (Alternative Nos. 4, 5 and 6) manufacturers were contacted for budgetary estimates for the respective equipment. Unit prices for various components and surcharges for electrical and instrumentation and control systems were applied based on experience from other similar projects. The construction estimates indicate that Alternative No. 4 (Nitrifying BAF's) has the highest estimated cost at approximately $3.67 million, and Alternative No. 1 (Nitrifying Activated Sludge, 2 AB's & 3 SC's) has the lowest estimated cost at approximately $1.79 million. However, Alternative No. 1 has reliability limitations, as noted above. Alternative No. 6 Nitrifying Trickling Filters has the second lowest estimated construction cost at $2.06 million.

3.2.4 Estimated Operating & Maintenance Costs of Biological Nitrification Alternatives

Operating requirements, including power, labor, chemicals and other consumables were estimated for each of the six alternatives. Power was estimated at $0.12 per kilowatt hour (kWhr); labor at $50 per hour, including City administrative overhead. Chemical costs were based on current local market rates. For Alternative Nos. 1, 2 and 3, which would treat the total flow to the entire WWTP, an annual average flow over the 20-year planning period was assumed at 3.8 mgd. For Alternative Nos. 4, 5 and 6, a constant flow of 2.55 mgd (as the required input to the MF/RO system) over the 20-year period was assumed. Alternative No. 3 (Nitrifying Activated Sludge with Chemically Enhanced Primary Treatment) has the highest estimated operating cost at approximately $314,000 per year. Alternative No. 6 (NTF's) has the lowest estimated operating cost at approximately $165,000 per year. The estimated operating cost of the other four alternatives range between $192,000 and $242,000 per year.

3.2.5 Quantitative Evaluation of Alternatives

The capital cost of a project includes both the initial construction cost plus engineering and construction management costs, required to implement the project.

The capital and annual O&M cost estimates presented herein are for comparative purposes only. These cost estimates were used to determine which alternative is the most cost-effective. Using the estimated capital and annual O&M costs for each alternative system, present worth values were developed to compare the life-cycle costs of the six alternatives. Present worth is defined as that amount of money it takes to fund the capital investment of a project, as well as its annual operating and maintenance costs, over a period of time, given the cost of money (interest) during the evaluation period. For this analysis, the time period used was 20 years and the interest rate was six percent. **Table 3-3** presents the results of this analysis.

Table 3-3
Summary of Economic Analysis of Biological Nitrification Alternatives

Component	***Alt No. 1 NAS (2&3) $1,000's***	***Alt No. 2 NAS (3&3) $1,000's***	***Alt No. 3 NAS&CEPT $1,000's***	***Alt No. 4 BAFs $1,000's***	***Alt No. 5 TSFF $1,000's***	***Alt No. 6 NTF $1,000's***
Estimated Construction Costs	$1,790	$3,310	$2,340	$3,670	$2,880	$2,060
Add 35% for Engineering and CM	$630	$1,160	$820	$1,280	$1,010	$720
Total Estimated Capital Cost	$2,420	$4,470	$3,160	$4,950	$3,890	$2,780
Estimated Annual O&M Costs	$202	$211	$314	$242	$192	$165
Present Worth of O&M Costs [(1)]	$2,320	$2,420	$3,610	$2,780	$2,200	$1,890
Total Estimated Present Worth Values	**$4,740**	**$6,890**	**$6,770**	**$7,730**	**$6,090**	**$4,670**

[(1)] PWF: i = 6% and n = 20 yrs

Alternative No. 6 has the lowest present worth value among the six alternatives analyzed. Alternative No. 1 has the next lowest present worth value by approximately 1.5%. Although Alternative No. 1 has the lowest estimated capital cost, it has significant reliability limitations in that it cannot consistently nitrify and meet the project's ammonia goal.

3.2.6 Qualitative Evaluation of Biological Nitrification Alternatives

In addition to capital cost, operating costs and present worth values, other qualitative factors were evaluated to aid in the selection of the best biological nitrification process. **Table 3-4** contains a tabular summary of qualitative factors and an assessment of how each alternative compares to each factor.

Table 3-4 Summary of Qualitative Evaluation of Biological Nitrification Alternatives						
Qualitative Factors	*Alt No. 1 NAS (2&3) $1,000's*	*Alt No. 2 NAS (3&3) $1,000's*	*Alt No. 3 NAS&CEPT $1,000's*	*Alt No. 4 BAFs $1,000's*	*Alt No. 5 TSFF $1,000's*	*Alt No. 6 NTF $1,000's*
Impact on Existing Facilities	Moderate	High	Moderate	Low	Low	Low
Ease of Operation	Good	Good	Moderate	Moderate	Good	Good
Ease of Implementation	Moderate	Difficult	Moderate	Good	Good	Good
Incrementally Expandable	Difficult	Difficult	Difficult	Moderate	Moderate	Moderate
Equipment Reliability	Good	Good	Good	Good	Good	Good
Process Reliability	Limited	Good	Limited	Good	Limited	Good
Proven Technology	Good	Good	Good	Good	Limited	Good
Process Complexity	Moderate	Moderate	Moderate	High	Low	Moderate
Power Demand	High	High	High	Moderate	Low	Lowest
Visual Impact	Low	Low	Low	High	Low	Moderate

Constructing additional process units to expand the existing biological treatment system will be disruptive to the City's WWTP, whereas a stand-alone system will not disrupt plant operations. All of the alternatives are relatively easy to operate, although the chemical addition system for Alternative No. 3 and the BAF backwashing system for Alternative No. 4 will require more operator attention.

Process reliability and technology for NAS alternatives are proven and understood. Performance data exist for plants operating in the NAS mode. Adequate operating data for nitrifying BAF's are also readily available, although less extensive than NAS systems. The nitrification processes of Alternatives 5 and 6 (TSFF and NTF's) can be designed to nitrify. However, limited operating data that support performance to the ammonia criterion of 2 mg/L have been provided by manufacturers of TSFF systems. NTF's have a longer operating record than TSFF systems, and that is why process reliability for NTF's systems is stated as "Good".

Visual impacts will be low, except for Alternative No. 4 BAF's, which have a high profile. Alternative No. 6 NTF's has a profile similar to the one-story building that will house the MF/RO system.

3.2.7 Conclusions and Recommendations

Based on the evaluation of the alternatives presented above, the following conclusions can be drawn:

1. Nitrifying Activated Sludge Alternative No. 1 does not provide reliable effluent quality of 2 mg/L ammonia for current average day flow rates.

2. Providing a reliable nitrifying activated sludge system by modifying the City's

activated sludge system will be highly disruptive and result in a high capital and operating cost, compared with other available, stand-alone alternatives.

3. Three stand-alone tertiary, biological nitrification alternatives are capable of meeting the 2 mg/L ammonia criterion. Biological activated filters and nitrifying trickling filters have more proven performance as stand-alone nitrification systems, than do submerged fixed film systems.

4. BAF's have a high equipment profile of about 25 feet; they also have the highest capital and operating cost.

5. Alternative No. 6 Tertiary NTF's appears to be the most cost-effective alternative that can meet the ammonia criterion of 2 mg/L..

6. Using a stand-alone nitrification system will avoid operational problems at the City's basic secondary treatment system during wet weather periods when it must accommodate high flows and still meets its NPDES permit requirements.

Based on the evaluations conducted and the information gained from a field trip to an existing, operating WWTP with NTF's, it is recommended that stand-alone NTFs be selected as the nitrification system to be used in the overall process scheme for the Benicia Water Reuse Project.

The NTF's units for Benicia would be approximately 42-ft in diameter and 15-ft high. Overall design criteria are shown in the **Table 3-5**, below.

Table 3-5
Summary of Facilities Required for Tertiary Nitrifying Trickling Filters

Item	***Description***
Feed pumps, including recycle (2)	2.0 mgd, each, approx 10 hp each
Trickling filters (2)	42 ft diameter x 12 ft media depth
Media	34,000 cf cross flow media
Process air blowers (8)	1,500 scfm, at 2-in H2O column (4 per filter)
Sodium hydroxide feed system, consisting of storage and 2 small chemical feed pumps and storage tank	Required for alkalinity control.

Secondary effluent will be pumped at a continuous flow rate (2.55 mgd) from the MPBs to the NTF's pump station wet well. The NTF pumps would be vertical turbine type mounted over a wet-well in between the two NTFs. They would not be enclosed but would be furnished with adequate noise reduction to meet City ordinance requirements at the WWTP fence line.

Since the nitrification process consumes about 7 mg of alkalinity per mg of ammonia converted, sodium hydroxide will be fed at the outlet of the NTF's to maintain alkalinity at the proper level. Sodium hydroxide would be stored in a fiberglass tank, mounted on a concrete pad outside. Full secondary containment would be provided.

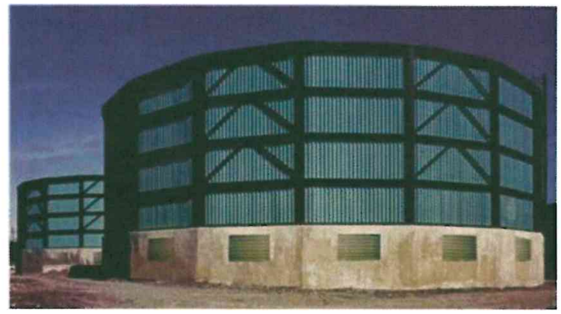

Figure 3-2
Typical Nitrifying Trickling Filters

Figure 3-2 shows a typical dual set of trickling filters, similar to the ones proposed for the Water Reuse Project.

3.3 Development and Evaluation of Advanced Treatment Systems

3.3.1 Evaluation of Partial Demineralization Systems

Computer simulation models of alternative partial demineralization schemes were run to determine the most cost-effective system that could process the City's effluent to meet the cooling water quality objectives. Technologies investigated in various combinations, included: granular media filtration, MF, NF, RO and EDR. As input to the demineralization analysis, it was assumed that ammonia would be biologically removed by nitrification down to approximately 2 mg/L, as discussed earlier in this section.

MF followed by RO was determined by computer simulations to meet all the requirements except for ammonia. Reducing the ammonia from about 0.3 mg/L to less than 0.2 mg/L will be met by breakpoint chlorination at the end of the treatment process after disinfection. Approximately 15% of the plant flow after MF will be routed around the RO system and blended with the RO permeate. Providing a 15% blend around reduces the cost of the RO system, and also provides the benefit of producing a more stable, less corrosive product water than if 100% RO treatment is used. **Figure 3-3** shows a process schematic of the MF/RO System.

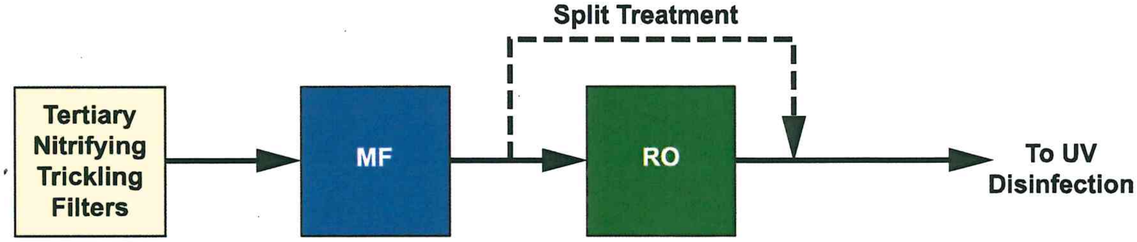

Figure 3-3
Process Schematic of the MF/RO System

Based on the above schematic the recycled water quality was projected to meet the water quality objectives as shown in **Table 3-6.**

Table 3-6
Comparison of Key Secondary Effluent Quality Parameters and Recycled Water Quality Limits and Projected Recycled Water Quality

Parameter	*Units*	*Benicia Secondary Effluent Water Quality*	*Cooling Water Quality Limits*	*Projected Recycled Water Quality*$^{(1)}$
ammonia	mg/L	30	<0.2	<0.2
bicarbonate	mg/L	190	104	37
chloride	mg/L	120	20	<20
phosphate	mg/L	2	3	0.5
silica	mg/L	22	17	4
hardness	mg/L	130	<200	23
TDS	mg/L	650	250	120

(1) Based on 15 % blend around the RO system and breakpoint chlorination

3.3.2 Pre-Treatment for Demineralization - Micro-Filtration System

In order to protect the RO membranes, micro-filtration is required. Typical MF systems processing secondary effluent will reject approximately 10% of the input flow. Hence, the output capacity of the MF system will be approximately 2.3 mgd. Motor operated strainers will be placed upstream of the MF's to protect them from residual particulates from the NTFs. MF systems are available in either the pressurized-type or the submerged, vacuum type. A pressurized MF system is recommended since it is more cost effective at the 2 mgd capacity. Horizontal, dry pit pumps will pump the influent to the MF system at discharge pressure of approximately 35 psi (approximately 80 feet of head). The MF system is backwashed at approximately 20-minute intervals using a combination of air and water. A compressed air supply system will be included to supply the necessary air. Citric acid and sodium hypochlorite will be used for enhanced backwash operation and the clean-in-place, membrane cleaning system.

Table 3-7 presents a summary of the design criteria for the micro-filtration system. **Figure 3-4** shows a typical pressure, micro-filtration system housed in a building.

Table 3-7
Summary of Micro-Filtration System Components

MF System Component	*Description/Criteria*
Design Output Flow Rate, mgd	2.3
Turbidity Process Performance, NTU	0.2 no > 5% in 24-hr
Reject Rate and Average Flow, %/mgd	10/0.25
Reject Flow Disposition	Recycled to Plant Headworks
Motor Operated Strainers	2 at 2 hp each
Supply Pumps (horizontal, dry-pit type	2 at 40 hp each
Design Flux Rate, gfd	25 to 40 (average)
MF Banks	Minimum of 3
Chemical Clean-in-Place System for MF Membranes	Acid & Hypochlorite Feed Pumps and Storage with Containment

Figure 3-4
Typical Micro-Filtration System
(courtesy of USFilter)

3.3.3 Partial Demineralization System - Reverse Osmosis

The reverse osmosis system will be designed for an output capacity of 1.7 mgd. When the RO treated water (known as permeate) is combined with the 15% "blend around" flow from the MF process, the total output will be 2.0 mgd. The RO system is estimated to have a recovery rate of approximately 85% of the influent flow. Hence, the reject or concentrate stream will be approximately 300,000 gpd. The RO system will include cartridge filters to protect the RO membranes from any solids carry over from the MF process and would also allow short periods of MF bypass for emergency operation. The RO system will be fed by two horizontal, dry pit pumps. Flow is boosted in a recycle step, internal to the RO system, by two booster pumps. **Table 3-8** below presents a summary of the RO System components. **Figure 3-5** shows a photograph of a typical RO system.

Table 3-8
Summary of Reverse Osmosis System Components

RO System Component	Description/Criteria
Design Output Flow Rate, mgd	1.7
Reject Rate and Flow, %/mgd	15/0.3
Reject Flow Disposition	Disposal to Existing Plant Outfall
Design Flux Rate, gfd	8 (average)
Cartridge Filters	40 inch
Low Pressure (35 psi) Supply Pumps (horizontal, dry-pit type)	2 at 40 hp each
High Pressure (125 psi) Booster Pumps (horizontal, dry-pit type)	2 at 150 hp each
Membrane Type	Polyamide
Chemical Anti-Scalant Feed System	Storage tank and metering pumps
Sulfuric Acid Feed System	Storage tank and metering pumps

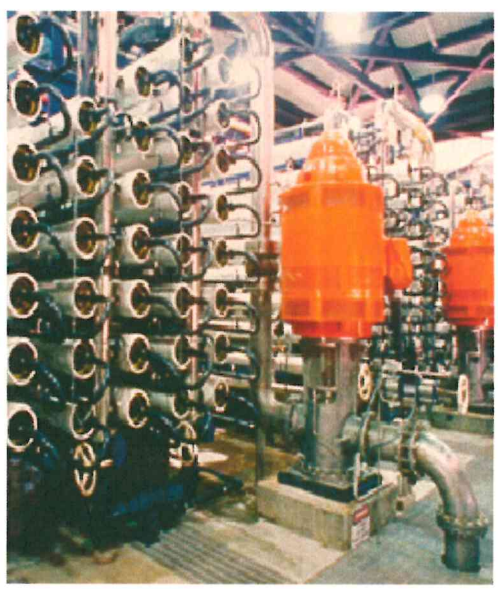

Figure 3-5
Typical RO System

3.4 Process Development and Analysis of Alternative Disinfection Systems

Alternative disinfection systems evaluated for the Benicia reuse project included chlorination using sodium hypochlorite and ultraviolet light disinfection. Recycled water from the proposed Water Reuse Treatment System must meet disinfection requirements for tertiary recycled water, proposed for use as cooling water supply, as contained in Title 22, Division 4, Chapter 3 of the California Code of Regulations (Title 22).

For either chlorination or UV disinfection, Title 22 requires that the median concentration of total coliform bacteria measured in the disinfected effluent shall not exceed 2.2 per 100 milliliters over the prior seven-day test period, not exceed 23 per 100 milliliters in more than one sample in any 30-day period, and never exceed 240 total coliform bacteria per 100 milliliters. Title 22 requires that a chlorine disinfection process must provide a CT (the product of chlorine residual and modal contact time) value of not less than 450 milligram-minutes per liter at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow. Generally, this results in a design hydraulic residence time of 120 min. Title 22 requires demonstration that alternative disinfection systems, such as UV, when combined with the filtration process, inactivate and/or remove 99.999 percent (5 log inactivation or removal) of the plaque-forming units of F-specific bacteriophage MS2, or polio virus in the wastewater. In addition, the micro-filtration process must meet the Title 22 turbidity performance requirements for micro-filtration which require that the filtered water does not exceed 0.2 NTU more than 5% of the time within a 24-hour period, and 0.5 NTU at any time.

The State Department of Health Services (DOHS) is responsible for approving UV disinfection systems. All UV disinfection systems proposed for water reuse in California must be validated under the 2003 NWRI/AWWARF Guidelines, which contain extensive design criteria. Three types of UV systems were reviewed for applicability to the Benicia water reuse project. Low Pressure, High Intensity (LPHI) was selected owing to energy efficiency and applicability to the size of this project.

For each disinfection alternative (high intensity UV vs. chlorination using sodium hypochlorite) conceptual designs were prepared and construction and O&M cost estimates were developed. For the UV alternative some chlorination is also required to prevent slime growths in the transmission pipeline. **Table 3-9** summarizes the present worth cost analysis of each alternative.

Table 3-9 Summary of Present Worth Analysis for Alternative Disinfection Systems		
	Chlorination $1,000s	**UV $1,000s**
Estimated Construction Costs	$980	$1,070
35% Allowance for Engineering, and CM Costs	$340	$375
Total Estimated Capital Costs	$1,320	$1,445
Estimated Annual O&M Costs	$77	$85
PW of O&M Costs	$880	$970
Total Estimated Present Worth	**$2,200**	**$2,415**

Based upon the accuracy of this cost analysis, both alternatives are judged equal in cost. Other qualitative factors, in particular water quality impacts, site impacts (owing to limited available space and allowances for future plant modifications) and ease of process control, favor UV over chlorination. Hence, UV was selected as the process alternative for disinfection.

Table 3-10 presents a summary of the facilities for the UV disinfection system for the project. The UV channel would be constructed of reinforced concrete and coated on the interior to prevent the potential growth of bacteria and pathogens on the walls of the channel. The UV channel will be covered to prevent the escape of the UV light, which is a hazard to eyesight. The electrical transformers and other equipment related to power and control will be located in a building. **Figure 3-6** shows a typical low pressure, high intensity UV module.

Table 3-10 Summary of Facilities for Low Pressure, High Intensity UV Disinfection System	
Item Description	**Criteria**
Number of Channels	1
Total Number of Banks, duty/standby	2/1
Modules per Bank	5
Lamps per Module	8
Total No. of Lamps	120
No. of Design Dose Lamps	80
No. of Redundant Lamps	40
Power Draw per Lamp	250 Watts
Max Power Draw Duty Lamps	20 kW
Average Power Draw	17kW
Channel Dimensions	
Length, ft	75
Width, in	21
Channel Depth, in	60

Trojan UV 3000 Plus Low Pressure High Intensity System – Typical Module

Figure 3-6
Example of UV Equipment

4

Section Four

Section 4
Regulatory Compliance and Pilot Testing

The concentrate (or reject) stream from the RO facility will be blended with the remaining Benicia WWTP discharge to the Carquinez Strait. The levels of constituents in the RO concentrate stream will be fives times higher than levels in the secondary effluent that will feed the MF/RO treatment system. Figure 4-1 shows a simplified flow diagram through the City's Wastewater treatment plan (WWTP) and the proposed Water Reuse Plant.

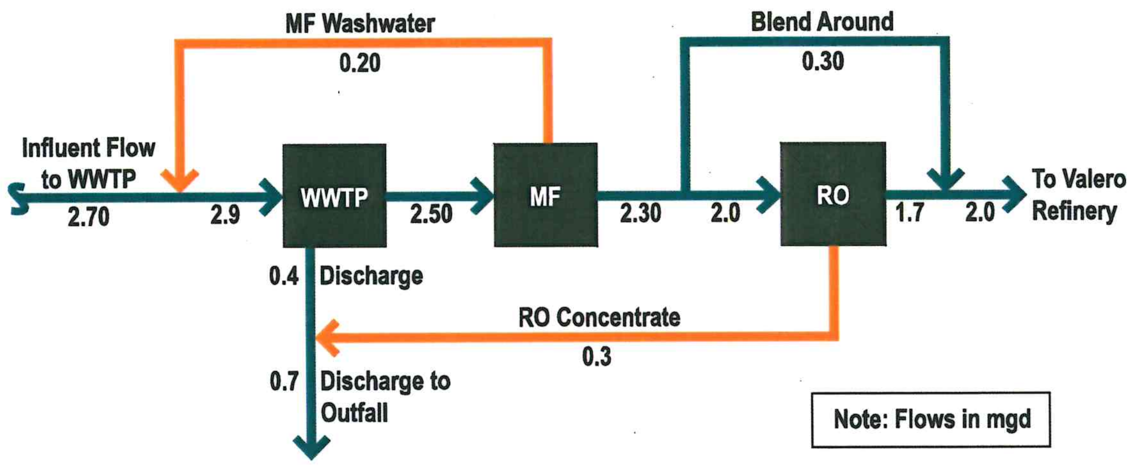

Figure 4-1
Typical Flow Balance for 2.0 mgd Water Reuse Plant

Initial planning level estimates have projected that up to 0.3 mgd of concentrate could be produced from the full-scale RO facility when operating at maximum capacity. That would be blended and discharged with the remaining approximately 0.4 mgd (minimum) of secondary effluent (i.e. a 43% blend).

Pilot-scale tests and laboratory analyses were performed to investigate the feasibility of the blended discharge (RO concentrate and Benicia WWTP effluent) meeting current NPDES discharge requirements and to characterize the following:

- Conventional Water Quality Parameters (BOD, TSS, pH, etc.)
- Trace Metals and other Priority Pollutants
- Acute and Chronic Toxicity

Several rounds of pilot tests were performed in order to generate RO permeate and concentrate streams using a pilot-scale RO treatment system. The RO treatment system was operated at high flux and high recovery rates, as listed in **Table 4-1**, to investigate the "worst case" scenario for the full-scale facility (i.e. to produce the highest concentration of constituents in the RO concentrate stream).

Table 4-1
Pilot-scale RO Treatment System Operating Parameters

Parameter (Unit)	*Operating Value**	*Typical for Secondary Effluent*
Recovery Rate (%)	85-87%	75-85%
Flux (gfd)	12	8-12
Feed Pressure (psi)	120-150	80-150

* Typical operating values from RO systems treating WWTP secondary effluent.

Figure 4-2 provides a schematic of the pilot-scale RO process. Samples of the RO feedwater (secondary effluent), RO permeate and RO concentrate were collected for lab analysis and toxicity tests.

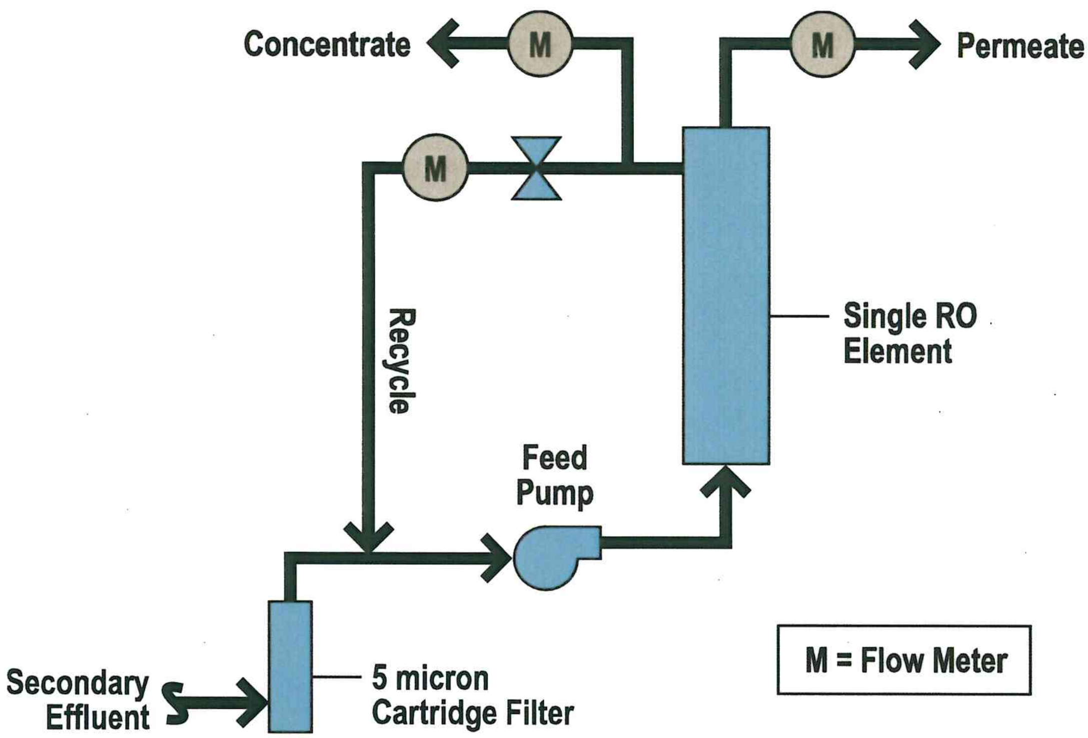

Figure 4-2
Pilot Scale RO Treatment System Schematic

The results of the pilot tests and lab analysis indicated that the effluent discharge blended with the concentrate should meet all permit requirements. Final results of the pilot studies will be presented to the RWQCB staff to update them on the status of the project. It is anticipated that only minor modifications will need to be made to the

NPDES permit when it is reissued in December 2007 to accommodate the project. The firm of EOA is coordinating regulatory compliance matters and negotiations.

The remaining portion of this section summarizes the pilot-scale results and existing NPDES requirements for regulatory compliance.

4.1 Water Quality Parameters

As expected, the results from the pilot testing indicated that levels of constituents (e.g. TDS) in the RO concentrate can be up to 5 to 7 times the levels of constituents entering the RO treatment system[1].

Table 4-2 presents ranges of general water quality data measured during pilot-scale tests conducted over the following time periods:

- Pre-test Demonstration - 10/6/04
- Round 1 - 10/12/04 to 10/15/04
- Round 2 - 1/18/05 to 1/24/05
- Round 3 - 11/12/04 to 11/19/04
- Round 4 - 6/6/06 to 6/7/06

Table 4-2
Pilot Testing General Water Quality Parameters[(1)]

Parameters	***Units***	***Secondary Effluent***	***RO Concentrate***
Alkalinity	mg/L-$CaCO_3$	210-300	1,300 – 2,000
pH	-	7.4-8.1	8.0-8.4
TDS	mg/L	550-710	3,100-5,800
BOD	mg/L	nd - 20	nd - 48
TSS	mg/L	nd - 10	nd - 38

(1) The data presented in table 2 are minimum and maximum values from analytical analysis following all rounds of the pilot-scale testing.

Mass balance equations were used to simulate a blend of 43% RO concentrate and 57% effluent to predict levels of contaminants in the blended discharge. From the results of the mass-balance analysis, it is anticipated that the actual combined concentrate and effluent blends of the full-scale facility will meet the following key treatment goals and limitations included in the current NPDES discharge permit listed below:

1 Five times the feedwater levels of constituents corresponds to the RO treatment system operating at a recovery ratio of 85% and greater than 90% concentration by the RO membrane element.

- Discharge pH shall not exceed 9.0 nor be less than 6.0
- Average BOD and TSS removal must be 85% or greater each calendar month
- Fecal Coliform Bacteria:
    1. Must not exceed a Most Probable Number (MPN) of fecal coliform bacteria of 200 MPN/100 ml (calendar month geometric mean)
    2. No more than ten percent (10 %) of all samples collected within each calendar month shall exceed a fecal coliform bacteria level of 400 MPN/100 ml.
- Discharge limits as listed in **Table 4-3**.

Table 4-3
Benicia WWTP NPDES Permit Discharge Limits

Conventional Pollutants	Units	Monthly Average	Weekly Average	Daily Maximum	Instantaneous Maximum
BOD	mg/L	30	45	60	--
COD	mg/L	25	40	50	--
TSS	mg/L	30	45	60	--
Oil & Grease	mg/L	10	--	20	--
Settleable Matter	mL/L-hr	0.1	--	0.2	--
Chlorine Residual	mg/L	--	--	--	0.0

It is important to note that the pilot-scale results were obtained with direct RO treatment of the secondary effluent and that the full-scale facility will provide lower levels of conventional pollutants (e.g. BOD and TSS) by utilizing nitrifying trickling filters and micro-filtration prior to RO treatment.

4.2 Trace Metals and Organics

The toxic substances regulated in the effluent discharge include trace metals, cyanide and two organic pollutants. **Table 4-4** lists discharge limits for toxic substances in the current Benicia effluent discharge permit.

Table 4-4
Benicia WWTP NPDES Toxic Substance Discharge Requirements

Constituent	Units	Daily Max	Monthly Average	Interim Daily Maximum	Interim Monthly Average
Cadmium	µg/L	17.4	5.7	--	--
Copper	µg/L	--	--	32	--
Lead	µg/L	45.7	17.3	--	--
Mercury	µg/L	--	--	1	0.087
Nickel	µg/L	70	30.2	--	--
Selenium	µg/L	--	--	31	--
Cyanide	µg/L	--	--	--	25
Dieldrin	µg/L	0.00028	0.00014	--	--
4,4-DDE	µg/L	0.00119	0.00059	--	--

To investigate the impact of adding RO concentrate to the effluent before discharge, the following analyses were performed during the pilot-scale testing to characterize levels of trace metals and priority pollutants of concern in the RO concentrate including the constituents listed above:

Daily Samples

Standard Minerals Package
Nitric Digestion for Metals (EPA 200.2)
Arsenic and Selinium by Hydride AA (SM 3114)
Cadmium, Copper, Lead, Nickel, Silver, Chromium, Zinc (EPA 200.8 ML)
Cyanide
Mercury (EPA 1631 ML)
Hexavalent Chromium (EPA 7196)
Fluoride (EPA 300.0)

Additional Samples Analyzed Each Round

PCB's (608.ML)
Full Dioxin EQ (EPA 1613)
PAHs (EPA 610.ML)
VOAs (EPA 624.ML)
BNA (EPA 625.ML)
Pesticides (EPA 614.ML & 632.ML)
Tributyltin

The levels of most organics in the WWTP effluent are consistently below the detection limit. The results from the pilot-scale testing showed that the levels of these organics were still under the detection limit in the RO concentrate.

Results from the pilot-scale testing also showed that the levels of trace metals in the RO concentrate are not anticipated to exceed discharge limits, as summarized in **Table 4-5**. The values listed in Table 4-5 are based on measured concentrations for secondary effluent and RO concentrate (maximum of two values for each pollutant) from the Round 4 testing, which is considered the most representative of future conditions. Three of the pollutants (Cu, Ni, CN) would trigger "reasonable potential" under the current water quality objectives. However, all three objectives will likely be superseded by site-specific objectives, which will be numerically higher, and no compliance difficulties are anticipated.

Table 4-5 Pilot Testing Trace Metal Results and NPDES Permit Limits				
Pollutant	***Secondary Effluent (RO Feed)***	***RO Concentrate***	***Blended Effluent***	***NPDES Limit***
	ug/l	**ug/l**	**ug/l**	**ug/l**
Cadmium	0.015	0.10	**0.05**	5.7
Copper	2.8	17.7	**9.2**	32
Lead	0.20	1.0	**0.54**	17.3
Mercury	0.0051	0.021	**0.012**	0.087
Nickel	2.2	16.6	**8.3**	30.2
Selenium	0.36	1.6	**0.88**	31.0

4.3 Acute and Chronic Toxicity

Representative samples of the effluent and RO concentrate were collected to perform three series of toxicity tests to demonstrate that the projected blend can meet NPDES discharge limits for acute and chronic toxicity. The first and third rounds tested the blended discharge of RO concentrate and Benicia WWTP effluent. The second round included a blend of the RO concentrate and Valero effluent.

4.3.1 Acute Toxicity

The Benicia WWTP NPDES permit acute toxicity effluent limits are expressed as follows:

The survival of bioassay test organisms in 96-hour bioassays of undiluted effluent shall be:
> *(1) an 11-sample median value of not less than 90 percent survival; and*
> *(2) an 11-sample 90th percentile value of not less than 70% survival.*

The acute toxicity tests consisted of parallel sets of static renewal tests, using the City's normal NPDES permit compliance test species: the freshwater Fathead Minnow, an estuarine fish species *Menidia beryllina* (Inland Silversides Minnow), and rainbow trout. The *Menidia* testing was conducted to test the hypothesis that the elevated (five to seven fold) and/or altered relative concentrations of non-toxic minerals (e.g., calcium, magnesium, chlorides) expected in the RO concentrate may be a source of toxicity to freshwater fish species such as the Fathead Minnow. In the *Menidia* testing protocol, the test solution (e.g., RO concentrate) has high quality salt added to bring concentrations up to that approximating seawater.

Three sets of acute toxicity tests were performed using blends of the RO concentrate and effluent; one during the first round of pilot-scale testing and two during the third round (3A & 3B). Results from the acute toxicity tests are presented in **Table 4-6**. The majority of tests showed 100% survival (i.e. no measurable toxicity). Survival results in the 43% RO concentrate blend were nearly identical to those in the 100% effluent for all three species tested. Based on these results, the blended discharge should meet all acute toxicity requirements.

Table 4-6			
Acute Toxicity Testing Results			
96-hour Static Renewal Test	***Round 1***	***Round 3A***	***Round 3B***
Fathead Minnow	mean % survival		
100% Benicia Effluent	100	100	100
9% RO Concentrate/Effluent Blend	Not tested	100	100
43% RO Concentrate/Effluent Blend	95	100	100
Rainbow Trout			
100% Benicia Effluent	Not tested	100	100
9% RO Concentrate/Effluent Blend	Not tested	100	100
43% RO Concentrate/Effluent Blend	Not tested	100	100
Inland Silversides Minnow			
100% Benicia Effluent	85	90	95
9% RO Concentrate/Effluent Blend	Not tested	100	90
43% RO Concentrate/Effluent Blend	100	95	95

4.3.2 Chronic Toxicity

Two series of chronic toxicity tests were performed. The first round of pilot testing used the Benicia NPDES permit compliance test species, *Mytilus*. Three additional species were added in the third round of testing to confirm the results and to determine whether other species might be more sensitive to the RO concentrate/effluent blend than *Mytilus*.

Compliance assessment for the City is determined by calculating chronic toxic units (TUc) as 100/EC25. The EC25 (EC=effective concentration) is a point estimate value obtained by applying statistical analysis to the toxicity data (a best fit line for the data), and is the modeled percent effluent concentration that would result in a 25% reduction in normal development of the *Mytilus* when compared to the Control.

The Benicia NPDES permit does not have enforceable effluent limits for chronic toxicity but instead has two "trigger values." Accelerated monitoring is required after exceeding either a three sample median trigger value of 10 chronic toxicity units (TUc) or a single sample maximum trigger of 20 TUc or greater. Further toxicity reduction evaluation (TRE) studies are required if the triggers continue to be exceeded during the accelerated monitoring.

The results from the first and third testing rounds are presented in **Table 4-7**. (Results from the second round are not presented since they primarily tested blends of Valero effluent and RO concentrate.) The majority of tests showed < 1 TUc results (i.e. no measurable chronic toxicity). Results in the 43% RO concentrate blend were nearly identical to those in the 100% effluent for all four species tested. Based on these results, the blended discharge should meet all chronic toxicity requirements.

Table 4-7
Chronic Toxicity Testing Results

Chronic Toxicity (TUc) Test	***Round 1 Growth***	***Round 3 % Survival***	***Round 3 Growth***
Blue Mussel (Mytilus)		**TUc Value**	
100% Benicia Effluent	7.6	Not tested	<1.0
9% RO Concentrate/Effluent Blend	Not tested	Not tested	<1.0
43% RO Concentrate/Effluent Blend	8.1	Not tested	1.6
Opossum Shrimp			
100% Benicia Effluent	Not tested	<1.0	<1.0
9% RO Concentrate/Effluent Blend	Not tested	4.2	2.0
43% RO Concentrate/Effluent Blend	Not tested	<1.0	1.2
Inland Silversides Minnow			
100% Benicia Effluent	Not tested	<1.0	<1.0
9% RO Concentrate/Effluent Blend	Not tested	<1.0	<1.0
43% RO Concentrate/Effluent Blend	Not tested	<1.0	<1.0
Giant Kelp			
100% Benicia Effluent	Not tested	<1.0	<1.0
9% RO Concentrate/Effluent Blend	Not tested	<1.0	<1.0
43% RO Concentrate/Effluent Blend	Not tested	<1.0	<1.0

NPDES permit trigger values require accelerated monitoring if results are greater than:
- 10 TUc units for a three sample median OR
- 20 TUc units for a single sample

4.4 Conclusions

Based on the results of the toxicity tests performed and the analytical results of the minerals, metals and other priority pollutants analyzed, the projected maximum blend of 43% RO concentrate with 57% secondary effluent should not result in any exceedances of the City's current or likely future NPDES permit requirements.

Blended effluent trace metals concentrations will increase due to the addition of the RO concentrate, however the total mass of metals (and organics) discharged to the Bay will remain the same. Blended effluent concentrations will typically be lower than shown by the mass balance calculations, given that effluent flows will be higher (i.e. more blending volume) than the conservative value used in the calculations.

5

Section Five

Section 5
Summary of Conceptual Design of Recycled Water Treatment Facilities

5.1 Process Schematic Diagram

Figure 5-1 presents the process schematic diagram for the recycled water treatment system. As shown in the schematic, secondary effluent flow will be equalized in the multi-purpose basins and conveyed to the nitrifying trickling filters (NTFs) for ammonia reduction. From the NTFs the water will be pumped through micro-filtration and reverse osmosis with a 15% blend of MF filtrate around the RO process. Before pumping to Valero the recycled water will undergo ultra-violet (UV) disinfection to meet regulatory requirements for use of recycled water in cooling towers. The high-lift pump station will convey the recycled water to Valero.

5.2 Conceptual Site Plan

Figure 5-2 shows the conceptual site plan and location of proposed recycled water treatment facilities on the City's WWTP site. The MF and the RO systems will be located either in separate buildings, as shown, or in one building. Such details will be analyzed and determined in the preliminary design phase. Chemical storage tanks will have full secondary containment and will be located for easy access for chemical deliveries. Disinfected water from the UV system will flow directly into the recycled water pump station.

5.3 Summary of Process Design Criteria

Based on the overall project design objectives and criteria presented in the prior sections, a summary of process design criteria was developed and is presented in **Table 5-1** below.

Table 5-1
Summary of Process Design Criteria

Item	***Criteria***
System Output Design Capacity, mgd	***2.0***
Secondary Effluent Pumping	
Pumping Range, mgd	1.0 to 3.5
Flow Equalization	
Existing Multi-Purpose Basins Storage Capacity, mg	0.4
Secondary Effluent Transfer Pump System	
Design Flow, mgd	2.55
Pumps, duty/standby, capacity	2.55
Pump Type	TBD
Biological Nitrification System	
Design Flow, mgd	2.55
Influent Ammonia Concentration, mg/L	30
NTF Effluent Ammonia Concentration, mg/L	2 to 3
Kinetic Temperature, degrees C	17
Recycle Ratio, %	50

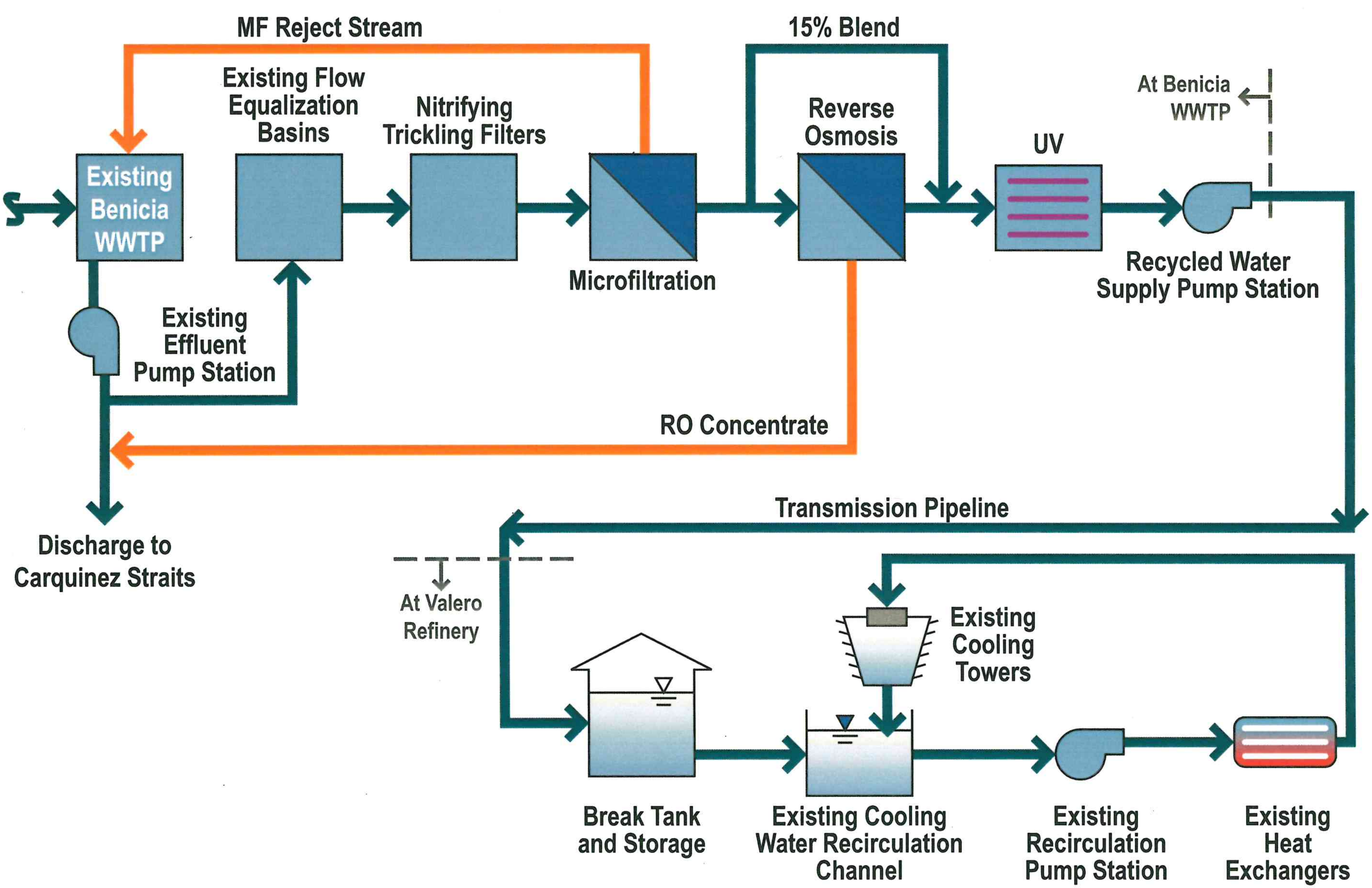

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Figure 5-1
Benicia Water Reuse Project - Process Schematic Diagram

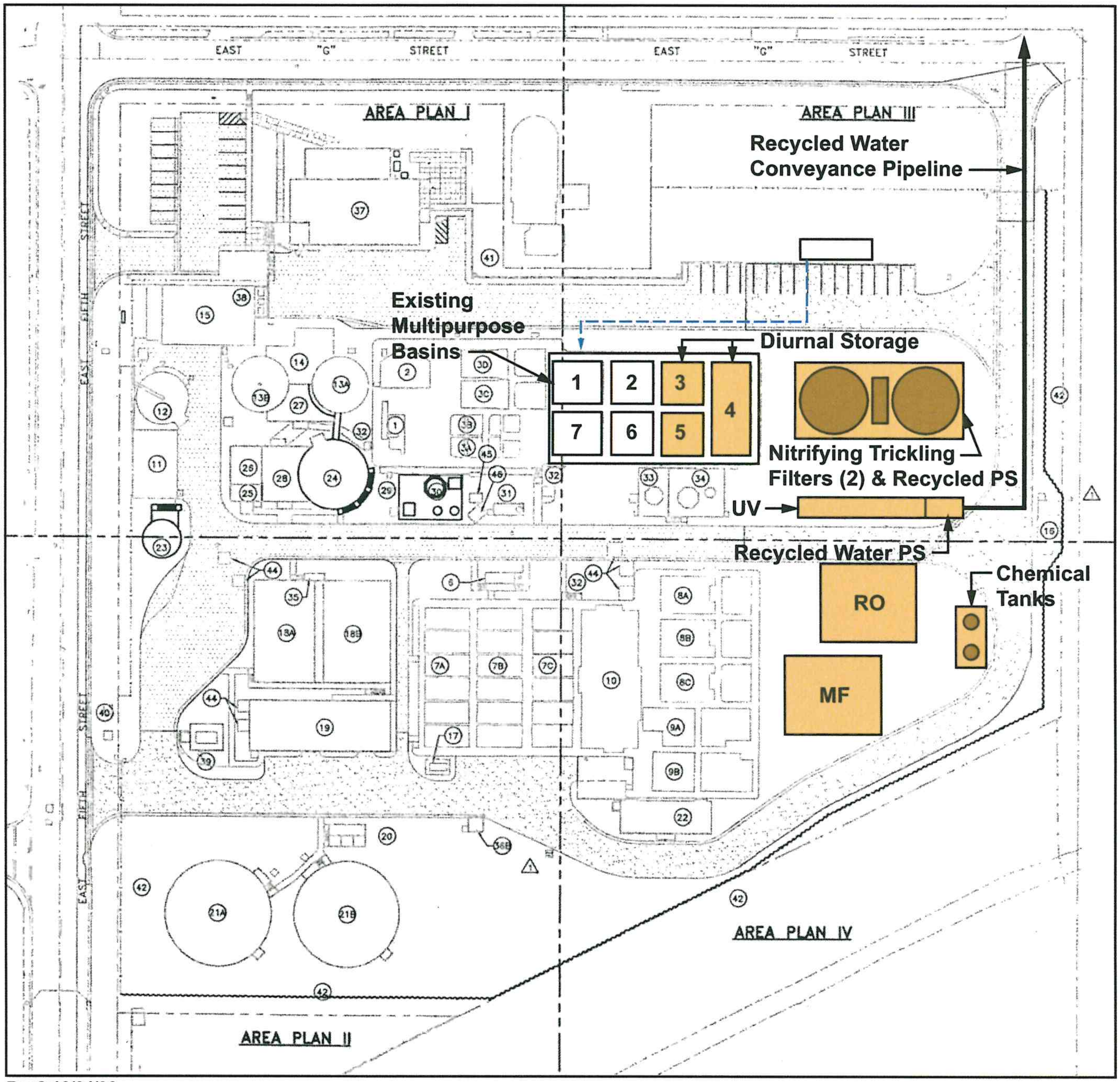

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Figure 5-2
Water Reuse Project Conceptual Site Plan

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Table 5-1
Summary of Process Design Criteria

Item	*Criteria*
Nitrifying Trickling Filters, number	2
Size, diameter x media depth, ft/ft	42/12
Media Type	Cross Flow
Media Specific Surface Area, sf/cf	45
Process Air Supply	
Centrifugal Blowers, number per NTF	4
Blower Capacity, scfm (each)	1,500
NTF Pumping System	
Design Flow, mgd (includes influent + recycle)	3.85
Pump Type	Vertical, Mix Flow
Number of Pumps, duty/standby	2/1
Pump Capacity, flow in gpm x head in ft	1350 x 25
Motor Horsepower, hp each	15
NTF Alkalinity Supply System	
Chemical Type	Sodium Hydroxide
Chemical Strength, %	30
Commercial Bulk Density at 30%, lbs Ca(OH)2/gal	3.4
Bio-Kinetic Replacement, lbs alkalinity per lbs ammonia	7.2
Estimated Caustic Dose, mg/L	52
Estimated Volume Caustic, gal/day	330
Estimated Storage Volume	
Micro-Filtration System	
Design Output Flow Rate, mgd	2.3
Turbidity Process Performance, NTU	0.2 no > 5% in 24-hr
Reject Rate and Average Reject Flow, %/mgd	10/0.25
Reject Flow Disposal: To Plant Head Works	
MF System Type	Pressure
Motor Operated Strainers	2 at 2 hp each
Supply Pumps (horizontal, dry-pit type)	2 at 40 hp each
Air Supply System: 15 hp compressor and receiver tank	
MF Membrane Type: Polypropylene or polyvinyldene fluoride (PVDF)	
Number of MF Modules	460 to 330(1)
Surface Area, sf/module	250 to 350(1)
MF Flux Rate, gfd	25 to 40(1)
MF Banks	Minimum of 3(1)
Chemical Clean-in-Place System for MF Membranes: Acid & Hypochlorite Feed Pumps and Storage with Containment	
Potential Manufacturers To Be Considered Include: Pall, Norit and US Filter	
Reverse Osmosis (RO) Demineralization System	
Design Output Capacity Flow Rate, mgd	1.7
MF Blend Around Flow Rate, mgd	0.3
Reject Rate and Flow, %/mgd	15/0.3
Reject Flow Disposal: To Existing Plant Outfall	
Cartridge Filters, number/size, inch	2/40
Average Design Flux Rate, gfd	8
RO Membrane Type	Polyamide
Number of RO Elements	520
Surface Area, sf/element	400
Number of RO Banks	(1)
Number of Elements/Bank	(1)
Low Pressure Supply Pump System	
Type of Pumps	Horizontal, Dry-Pit
Number of Pumps, duty/standby	1/1
Design Pressure, psi	35
Motor Horsepower, hp ea	40

Table 5-1 ***Summary of Process Design Criteria***	
Item	***Criteria***
High Pressure Booster Pump System	
Type of Pumps	Horizontal, Dry-Pit
Number of Pumps, duty/standby	1/1
Design Pressure, psi	125
Motor Horsepower, hp ea	150
Chemical Anti-Scalant Feed System	
Sulfuric Acid Feed System	
UV Disinfection System	
Type of UV System: Low Pressure, High Intensity	
Number of Channels	1
Total Number of Banks, duty/standby	2/1
Modules per Bank	5
Lamps per Module	8
Total No. of Lamps	120
Power Draw per Lamp	250 Watts
Max Power Draw Duty Lamps	20 kW
Average Power Draw	17kW
UV Channel Dimensions, LxWxD, ft/in/in	75/21/60
Potential Manufacturers to be Considered: Trojan, IDI/Ondeo and Wedeco	
Breakpoint Chlorination System	
Blended RO/MF Ammonia Concentration, mg/L	0.4
Target Recycled Water Ammonia Concentration, mg/L	0.1
Stoiciometric Reduction of Ammonia by Chlorine, mg/mg	7.5
Desired Chlorine Residual, mg/L	2
Estimated Chlorine Dose, mg/L	5
Form of Chlorine Chemical: Sodium Hypochlorite	
Commercial Strength of Sodium Hypochlorite, %	12.5
Full Chemical Strength of Sodium Hypochlorite at 12.5% is one lb of chlorine per gal	
Estimated Volume of Hypochlorite, gal/day	83
Estimated Hypochlorite Storage Volume Required, gal	
Hypochlorite Feed Pumps, duty/standby	1/1
Hypochlorite Feed Pump Capacity, gal/hr	2 to 5
Design Hydraulic Residence Time (HRT), min	10
Required Contact Tank Volume, gallons	14,000

(1) Depends on manufacturer

Section 6

Section 6
Recycled Water Conveyance System

6.1 Introduction

The conveyance system that will deliver recycled water from the Benicia WWTP site to the Valero Refinery will consist of a pump station at the WWTP, a pipeline approximately 14,000 feet in length and a "break tank" storage facility at the Refinery. Beginning at the WWTP the pipeline will travel from a new, high-lift recycled water supply pump station (RWSPS) to the Valero "off site" dock line right-of-way in the vicinity of East 7th Street and "H" Street. The pipeline will follow the abandoned Valero dock lines northerly for about 9,000 feet to the refinery property line. Within the refinery the pipeline will follow Avenue "E" South, then up a vertical rise (known as a "waterfall") to Avenue "F" to the cooling towers.

The existing Valero dock lines are attached to above-grade structural steel frames, known as "sleepers." An evaluation compared the cost of rehabilitating existing dock lines versus constructing new piping and it was determined that it was more cost-effective and reliable to install a new, 14-inch pipeline, rather than rehab portions of the existing dock lines. Within the refinery, new pipeline will be constructed on vertical extensions to the existing pipeline "sleepers" that parallel Avenues "E" and "F." The break tank would be located near the cooling towers. The capacity of the break tank is to be determined, but it is anticipated that the capacity will be equal to 4 to 6 hours of recycled water flow.

6.1.1 Overview of Conveyance Pipeline Profile

Valero provided CDM with copies of plan and profile drawings of the "off-site" dock lines as well as information about the pipe material, pressure class and wall thickness. Valero also provided information on the elevation of the existing pipeline sleepers within the refinery. Using this information, CDM developed a preliminary profile of the pipeline from the City's WWTP to the cooling towers. The profile begins at the City's WWTP near elevation zero and reaches a high point approximately one mile northerly along the alignment at approximate elevation 201. The pump station at the WWTP will be located at approximate elevation zero. Hence the static lift will be about 200 ft. The terminal point of the pipeline will be at the cooling water recirculation channel located adjacent to the refinery cooling towers at elevation 95.

6.2 Recycled Water Pump Station

The recycled water pump station will consist of three (2 duty/1 standby), vertical turbine pumps mounted over a clearwell. The clearwell will be constructed of reinforced concrete and the pumps will not be housed in a building. **Table 6-1** presents the major components and design criteria for the pump station.

6.2.1 Instrumentation, Control, Monitoring and Sampling

6.2.1.1 Pump System Control

The RWSPS pumps will be automatically controlled by a programmable logic controller (PLC), based on water level in the clearwell. In that way, RWSPS will match the production rates of the water reuse treatment plant, which will be controlled to match daily demand. The pumps will also be able to be controlled to pump at a selected flow rate by setting a specific rate through a PLC. Manual pump start and stop and speed control will also be provided at the PLC.

Control interlocks with other systems will be as follows:

- All of the RWSPS pumps will be automatically stopped on high level in the break tank at Valero to avoid overfilling the tank.
- All of the RWSPS pumps will be automatically stopped on high micro-filtration effluent turbidity conditions.
- All of the RWSPS pumps will be automatically stopped on detection of critical alarm conditions at any of the upstream treatment processes.
- Under any of the hydraulic or process performance alarm conditions that would shut down the pumps, the recycled water would be routed to the City's outfall until the alarm conditions have been addressed and cleared.

6.2.1.2 Monitoring

The following monitoring provisions will be incorporated into the pump station design:

- Water level in the clearwell will be continuously monitored using an ultrasonic level sensor, with separate float switches for high and low level alarms in the event of failure of the level sensor. The water level signal will be used for pump control as described above.
- A magnetic flow meter will be provided on the pump discharge header to measure pump flow rate. The flow signal will be used for regulatory and recycled water inventory recordkeeping, for RWSPS monitoring and for pump control as described above.
- A pressure transducer will be provided on the recycled water discharge header to continuously measure header pressure for the purposes of monitoring pump operation and head conditions in the transmission system.
- A locally indicating pressure gauge will be provided on the discharge header and on each pump discharge.

6.2.1.3 Sampling

A refrigerated automatic composite sampler may be required for regulatory sampling. The RWQCB may require the City to sample and report the quality of recycled water leaving the City's property. The sampler would draw from the recycled water discharge header and would be flow paced from the RWSPS flow meter. An on-line ammonia analyzer will be provided to warn of ammonia concentrations exceeding the water quality requirements.

Table 6-1
Recycled Water Pump Station
Preliminary Design Criteria

Component	Units	Criteria
System Pumping Requirements		
Design Capacity	mgd	2.0
Design Capacity	gpm	1,400
Design Total Dynamic Head	ft	250
Static Head	ft	200
Pump Units		
Type		Vertical Turbine
Number, Total/Duty/Standby		3/2/1
Design Capacity per Pump		700
Design TDH per Pump	ft	255
Min. Efficiency at Design Point	%	82
Stages per Pump	No.	4
Pump Operation		Variable
Minimum Speed	rpm	TBD
Pump Motors		
Type		TEFC, w/ noise control enclosures
Size, each unit	hp	60
Drive Type		VFD
Synchronous Speed	rpm	1,800
Power Supply		480-V/3-phase/60Hz
Pump Discharge Piping		
Diameter	inch	8
Velocity at Design Flow	fps	4.43
Pumps Discharge Header Piping		
Diameter	inch	14
Velocity at Design Flow	fps	2.90
Discharge Flow Metering		
Type		Magnetic or Sonic
Size	inch	10
Velocity at Design Flow Rate	fps	5.67

6.3 Recycled Water Conveyance Pipeline

Figure 6-1 presents a map of the recycled water transmission pipeline and **Figure 6-2** show the preliminary hydraulic profile. **Table 6-2** presents the details of the pipeline for each segment from the City's WWTP to the Valero Refinery. The pipe would be constructed of cement mortar lined steel pipe. Where the pipe is buried, it will be cement mortar coated and taped. The coating system for pipe installed on sleepers is to be determined. Joints will be welded.

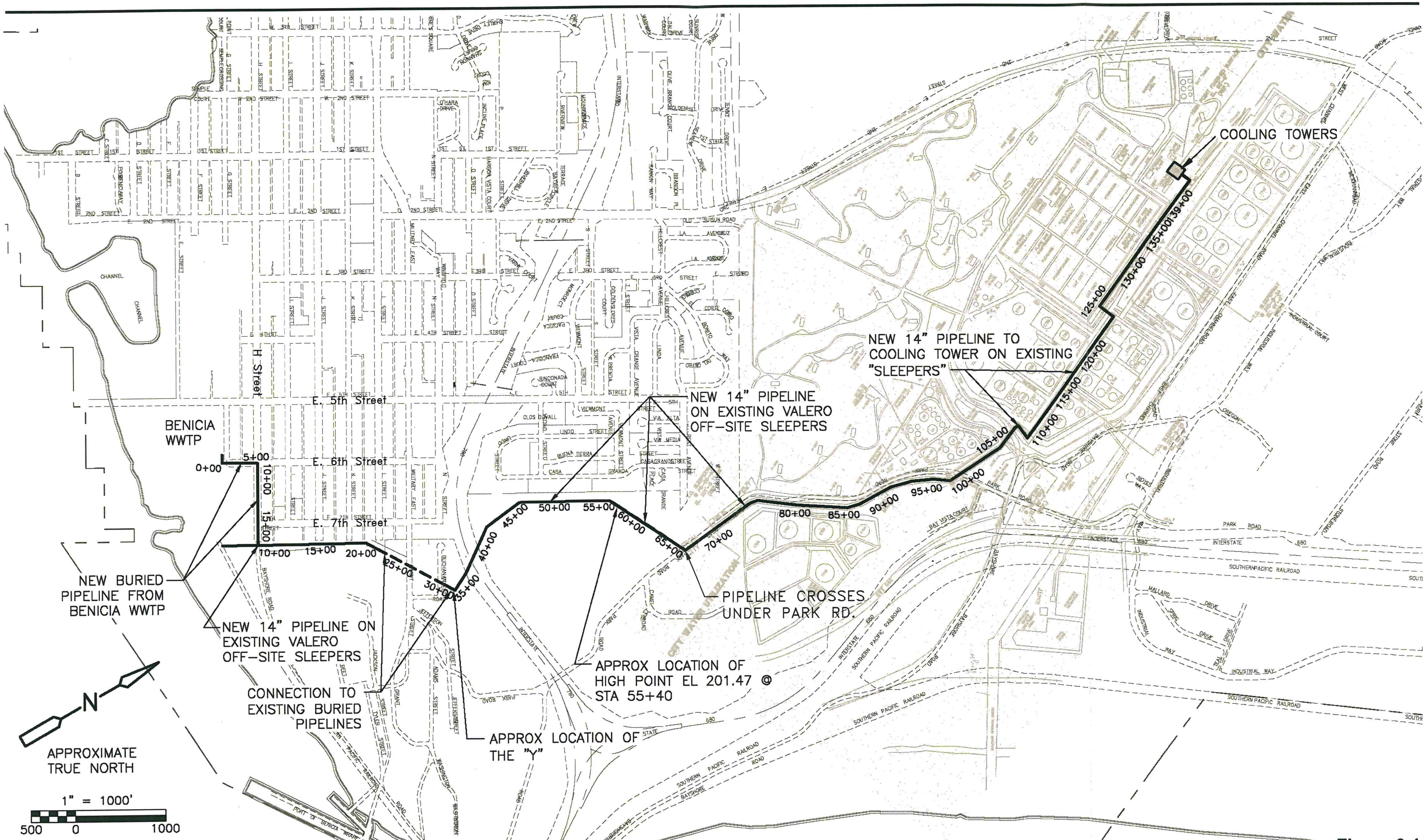

Figure 6-1
City of Benicia Water Reuse Project Pipeline Route Map

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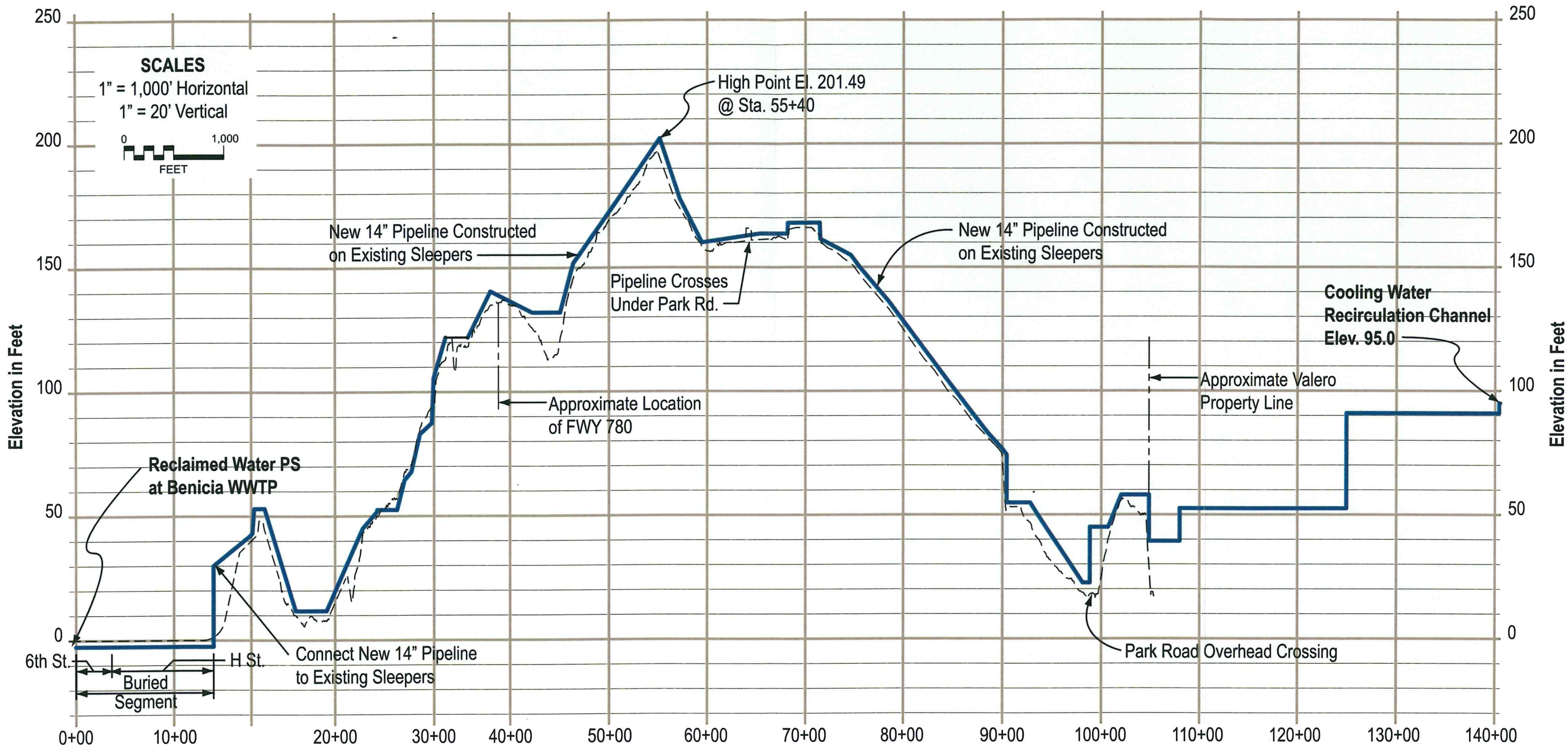

Figure 6-2
City of Benicia Water Reuse Project Preliminary Hydraulic Profile of Recycled Water Pipeline

Line isolation valves will be installed about every 2,000 feet to isolate sections for maintenance and/or repairs. Blow down valves will be located at low points to either drain the line or to "blow down" residual solids, which are unlikely to occur given the high level of treatment. Air inlet and vacuum relief valves will be installed at critical high points to control the potential effects of high pressure and hydraulic transients.

Table 6-2
14-Inch Recycled Water Conveyance Pipeline from Benicia WWTP To Valero Cooling Towers

System Component	From Sta	To Sta	Estimated Quantities, ft
Segment No. 1: Sta 0+0 @ Benicia WWTP to Sta 17+75 @ connection to sleepers. Construct new buried pipeline	0	1775	1775
Segment No. 2: Sta 11+45 @ start of sleepers to Sta 24+60 @ start of existing, buried 12-in lines. Construct new pipe on existing sleepers	1145	2460	1315
Remove Dock Line No. 3 from sleepers in Segment No. 2			1315
Segment No. 3: Sta 24+60 to Sta 30+00 end of existing, buried 12-in lines. Rehabilitate and connect to 2, existing 12-in lines	2460	3000	540
Segment No. 4: Sta 30+00 to Sta 32+20 at the "Y" plus additional 30 ft. Construct new, 14-in pipe on existing sleepers	3000	3220	220
Remove Dock Line No. 3 from sleepers in Segment No. 4			220
Segment No. 5: Sta 34+68 at the "Y" to Sta 42+15, end of where existing 12-in DL has been removed. Construct new, 14-in pipe on existing sleepers	3468	4215	747
Segment No. 6: Sta 42+15 to Sta 85+20, end of existing, abandoned 12-in DL. Construct new, 14-in pipe on existing sleepers	4215	8520	4305
Remove abandoned pipe from sleepers in Segment No. 6			4305
Segment No. 7: Sta 85+20 to Sta 105+00, approximate Valero PL. Construct new, 14-in pipe on existing sleepers.	8520	10500	1980
Segment No. 8: Sta 105+00 to Sta 140+00, approximate location of cooling towers. Construction new, 14-in pipe on extensions to existing sleepers.	10500	14000	3500
6-inch Air Inlet and Vacuum Release Valves Located at High Points			4
2-inch Air Inlet and Vacuum Release Valves			5
6-inch Blow Down Valves (BV's) Located at Low Points			6
In-Line Isolation Valves (BV's) Located at 2,000 ft intervals			7

7

Section Seven

Section 7
Estimated Project Costs

The purpose of this section is to briefly summarize the estimated capital and annual operating costs for the Water Reuse project. Before presenting these costs, the assumptions used in developing these costs are described.

7.1 Bases for Cost Estimates

7.1.1 Construction Cost Estimates

- **Foundations** – Owing to the poor soil conditions (Bay mud) in the area available for the Project, it will be necessary to place new structures on pile foundation systems. Based on review of the Geotechnical Engineering and Environmental Services Report, dated 15 July 1997 and prepared by Harza Engineers for the City's 1998 WWTP Improvement Project, pre-cast concrete piles, driven to an approximate depth of 70 feet have been assumed. Conceptual design estimates were made of the number of piles per structure, plus mobilization and demobilization.

- **Structural** – Water bearing tanks, channels, wet wells and the like were assumed to be constructed of cast-in-place reinforced concrete.

- **Civil** – Civil site work costs were estimated at 20% of structural costs (excluding foundation costs) to cover site preparation, grading, paving and minor site piping. Major piping was estimated separately based on unit prices from other applicable projects.

- **Mechanical –** Budgetary estimates for mechanical equipment were obtained from vendors and/or were based on experience from other recent similar projects.

- **Electrical** – Electrical costs were estimated at between 30 to 50 percent of the mechanical equipment cost based on complexity of the systems and on experience with construction of similar systems. Site electrical power was separately estimated based on supplying power to the Water Reuse Project through the plant's existing electrical service and running separate conduit and cable to a new substation at the site of the project. Power consumption for the project would be separately metered.

- **Instrumentation** - Instrumentation will be required for process monitoring and control and for connection to the plant SCADA system. Typical instrumentation includes monitoring of water levels, flow rates, total dissolved solids, ammonia, turbidity, chlorine residual, pH, UV transmittance, and others. Monitoring of data available from the manufacturer-furnished control panels will also be provided. The instrumentation costs were estimated at 20 percent of mechanical equipment cost.

7.1.2 Operating and Maintenance Cost Estimates

- **Electrical Power Cost** – Electrical power costs used were $0.12/kWhr, which is based on the average unit price for power at the WWTP for one winter month and one summer month.

- **Labor Cost** – Labor cost was assumed at $50/hr, which includes City's normal general and administrative overhead. Administrative labor costs were estimated at 15% of the direct operations and maintenance costs for management and supervision.

- **Equipment Repair and Replacement** – An allowance of two percent (2%) per year of the estimated construction cost of major mechanical and electrical equipment was made to establish a sinking fund to repair and replace major items of equipment.

- **Chemicals** – Costs of sodium hypochlorite, sodium hydroxide and sodium bisulfite were obtained from City WWTP staff for actual cost paid for these chemicals. Cost for other chemicals (sulfuric acid, ascetic acid and antiscalants) were obtained from suppliers.

- **Other Consumables** – Cost of other consumables, such as replacement membranes, cartridge filters, UV lamps and ballasts were obtained from the respective equipment vendors. Those estimated costs are presented within the estimate of each system.

- **Special Maintenance** – Many WWTPs contract out for special maintenance services for electrical and instrumentation systems. An allowance for a special maintenance contract was made in the amount of $50,000, based on experience from a similar plant.

7.2 Estimated Construction and Capital Costs

Using the cost bases describe above, construction costs were estimated for the various unit processes. The capital cost of a project includes both the initial construction cost plus all "soft costs" that are required to implement the project. These soft costs include: engineering, construction management, administration, environmental compliance, acquisition of permits and financing costs. Other assumptions used in developing the project capital cost estimates are:

- CDM's previous capital cost estimates are presented in the technical memoranda that are found in the appendix. These estimates were prepared in late 2004 and early 2005. They have been adjusted to account for higher than anticipated construction cost escalation in 2005 and 2006.

- Estimates include the cost for a "break tank" at Valero sized for 6 hrs of storage at the water reuse plant flowrate.

- Estimates include 25% for engineering design and construction management, 25% for contingencies, and $1 million for the preliminary engineering, water quality testing, and environmental planning costs that will be completed prior to the start of engineering design.

- The project will be bid in May, 2008.
- The contractor will price the project to the mid-point of construction (May 2009).
- Construction cost escalation between October 2006 and May 2009 will range between 6% and 12% annually.

Tables 7-1, 7-2, and 7-3 present a summary of the estimated capital costs for projects with production capacities of 2.0, 1.5 and 1.0 mgd, respectively.

Table 7-1
Summary of Estimated Capital Cost
2.0 mgd Water Reuse Project

Component	Cost ($ millions)
Microfiltration/Reverse Osmosis Systems	$11.05
Civil/Electrical Site Work	$0.87
UV Disinfection System	$1.18
Recycled Water Pump Station	$0.54
Pipeline	$2.26
Nitrifying Trickling Filters	$2.29
Valero break tank, 0.5 MG	$0.50
Subtotal	$18.68
Engineering and CM at 25%	$4.67
Subtotal	$23.35
Contingency at 25%	$5.84
Costs for preliminary engineering, water quality testing, and environmental planning	$1.00
Total Cost based on Oct. 2006	$30.18
Total Capital Cost, assuming 6% annual inflation to mid-point of construction in May 2009 (2.5 yrs)	**$34.92**
Total Capital Cost, assuming 12% annual inflation to mid-point of construction in May 2009 (2.5 yrs)	**$40.14**

Table 7-2
Summary of Estimated Capital Cost
1.5 mgd Water Reuse Project

Component	Cost ($ millions)
Microfiltration/Reverse Osmosis Systems	$8.91
Civil/Electrical Site Work	$0.84
UV Disinfection System	$1.09
Recycled Water Pump Station	$0.50
Pipeline	$2.07
Nitrifying Trickling Filters	$2.06
Valero break tank, 0.38 MG	$0.38
Subtotal	$15.84
Engineering and CM at 25%	$3.96
Subtotal	$19.80
Contingency at 25%	$4.95
Costs for preliminary engineering, water quality testing, and environmental planning	$1.00
Total Capital Cost based on Oct. 2006	$25.75
Total Capital Cost, assuming 6% annual inflation to mid-point of construction in May 2009 (2.5 yrs)	**$29.80**
Total Capital Cost, assuming 12% annual inflation to mid-point of construction in May 2009 (2.5 yrs)	**$34.25**

Table 7-3
Summary of Estimated Capital Cost
1.0 mgd Water Reuse Project

Component	Cost ($ millions)
Microfiltration/Reverse Osmosis Systems	$6.13
Civil/Electrical Site Work	$0.69
UV Disinfection System	$1.00
Recycled Water Pump Station	$0.50
Pipeline	$2.07
Nitrifying Trickling Filters	$1.67
Valero break tank, 0.25 MG	$0.25
Subtotal	$12.31
Engineering and CM at 25%	$3.08
Subtotal	$15.38
Contingency at 25%	$3.85
Costs for preliminary engineering, water quality testing, and environmental planning	$1.00
Total Capital Cost based on Oct. 2006	$20.23
Total Capital Cost, assuming 6% annual inflation to mid-point of construction in May 2009 (2.5 yrs)	**$23.40**
Total Capital Cost, assuming 12% annual inflation to mid-point of construction in May 2009 (2.5 yrs)	**$26.90**

7.3 Estimated Annual Operation and Maintenance Costs

The annual O&M costs of the project include power, labor, chemicals, and replacement of consumables (e.g., membranes, UV lamps, etc). Labor estimates were based on experience with other operations at plants, available guidelines and discussions with existing Benicia Plant operations staff. The replacement costs for major consumables were based on manufacturers' recommendations and experience with other projects.

A summary of estimated annual O&M costs is presented in **Tables 7-4, 7-5, and 7-6.**

Table 7-4
Estimated Annual O&M Costs of 2.0 mgd Water Reuse Project

Item	***NTF's***	***MF***	***RO***	***UV***	***Pumping***	***Admin***	***Totals***
Chemicals	$91,000	$56,300	$68,600	$19,600	$0	$35,300	$270,800
Power	$26,700	$44,500	$151,000	$17,700	$108,300	$52,200	$400,400
Consumables	$0	$63,500	$61,100	$16,700	$0	$21,200	$162,500
Equipment R/R	$18,400	$20,100	$23,200	$15,600	$9,200	$13,000	$99,500
Labor	$28,800	$68,800	$48,800	$38,000	$23,900	$31,200	$239,500
E and I&C Maint.	$0	$20,000	$10,000	$11,000	$3,000	$6,000	$50,000
Total	**$164,900**	**$273,200**	**$362,700**	**$118,600**	**$144,400**	**$158,900**	**$1,222,700**

Table 7-5
Estimated Annual O&M Costs of 1.5 mgd Water Reuse Project Capacity

	NTF's	***MF***	***RO***	***UV***	***Pumping***	***Admin***	***Totals***
Chemicals	$68,300	$42,200	$51,500	$14,700	$0	$26,500	$203,200
Power	$20,000	$33,400	$113,300	$13,300	$81,200	$39,200	$300,400
Materials	$0	$47,600	$45,800	$12,500	$0	$15,900	$121,800
Equipment R/R	$17,460	$20,080	$17,300	$14,300	$8,500	$11,600	$89,240
Labor	$28,800	$68,800	$48,800	$38,000	$23,900	$31,200	$239,500
E and I&C Maint	$0	$20,000	$10,000	$11,000	$3,000	$6,000	$50,000
Total	**$134,560**	**$232,080**	**$286,700**	**$103,800**	**$116,600**	**$130,400**	**$1,004,140**

Table 7-6
Estimated Annual O&M Costs of 1.0 mgd Water Reuse Project Capacity

	NTF's	***MF***	***RO***	***UV***	***Pumping***	***Admin***	***Totals***
Chemicals	$45,500	$28,200	$34,300	$9,800	$0	$17,700	$135,500
Power	$13,400	$22,300	$75,500	$8,900	$54,200	$26,100	$200,400
Materials	$0	$31,800	$30,600	$8,400	$0	$10,600	$81,400
Equipment R/R	$15,960	$20,080	$11,600	$12,700	$8,500	$10,300	$79,140
Labor	$28,800	$68,800	$48,800	$38,000	$23,900	$31,200	$239,500
E and I&C Maint	$0	$20,000	$10,000	$11,000	$3,000	$6,000	$50,000
Total	**$103,660**	**$191,180**	**$210,800**	**$88,800**	**$89,600**	**$101,900**	**$785,940**

Figure 7-1 shows graphic distribution of the estimated annual O&M cost for a 2 mgd capacity plant. Labor costs make up approximately 33% of the estimated O&M costs and power and chemicals each make up about 25%.

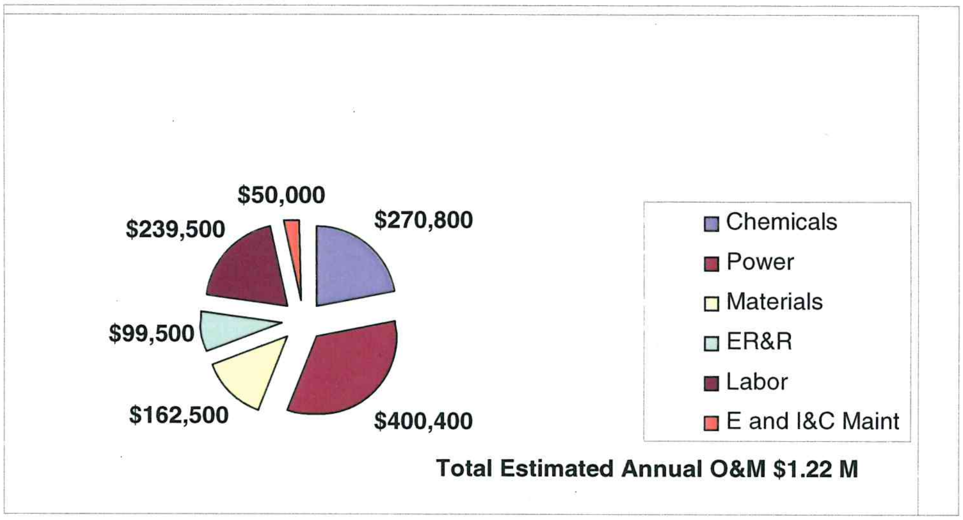

Figure 7-1
Distribution of Estimated O&M Costs for 2.0 mgd Capacity Project

7.4 Estimated Cost of Recycled Water Production

Based on the estimated O&M costs presented in **Tables 7-4, 7-5, and 7-6**, the unit cost of producing and delivering recycled water to Valero was calculated for plant productivity ratios of 75% and 100%. Labor, materials, equipment repair/replacement and special electrical & instrumentation cost remain constant and independent of flow. Chemical and electrical power costs vary nearly directly proportional to flow. Based on those assumptions, the unit costs are shown in Table 7-7. For example, the unit cost for a 2 mgd plant varies from $530/acre foot (AF) at 100% productivity (basically, 24/7/365) up to $630/AF at 75% productivity. These estimates do not include the amortization of capital costs.

Table 7-7
Cost of Producing Recycled Water[(1)]
($/Acre ft)

Percent Productivity	*2 mgd Plant Production Capacity*	*1.5 mgd Plant Production Capacity*	*1.0 mgd Plant Production Capacity*
100[(2)]	$546	$597	$701
75[(3)]	$630	$689	$809

[(1)] Do not include amortized capital costs.
[(2)] 24/7/365 operation at 100% capacity.
[(3)] For example, 24/7/365 at 75% capacity.

8

Section Eight

Section 8
Project Schedule

8.1 Project Schedule

Figure 8-1 contains an updated project milestone schedule. As shown therein, the ongoing CEQA compliance process is the primary current project activity. The CEQA consultant projects that a mitigated negative declaration can be certified about February 2007. Until such certification, it may not be prudent to move forward with final design as there could be changes. Although not shown, another major issue affecting the schedule is the availability of project funding, which will determine the design output capacity of the Project.

Based on the information currently available it appears that the project could be operational and delivering recycled water in the second quarter of 2010.

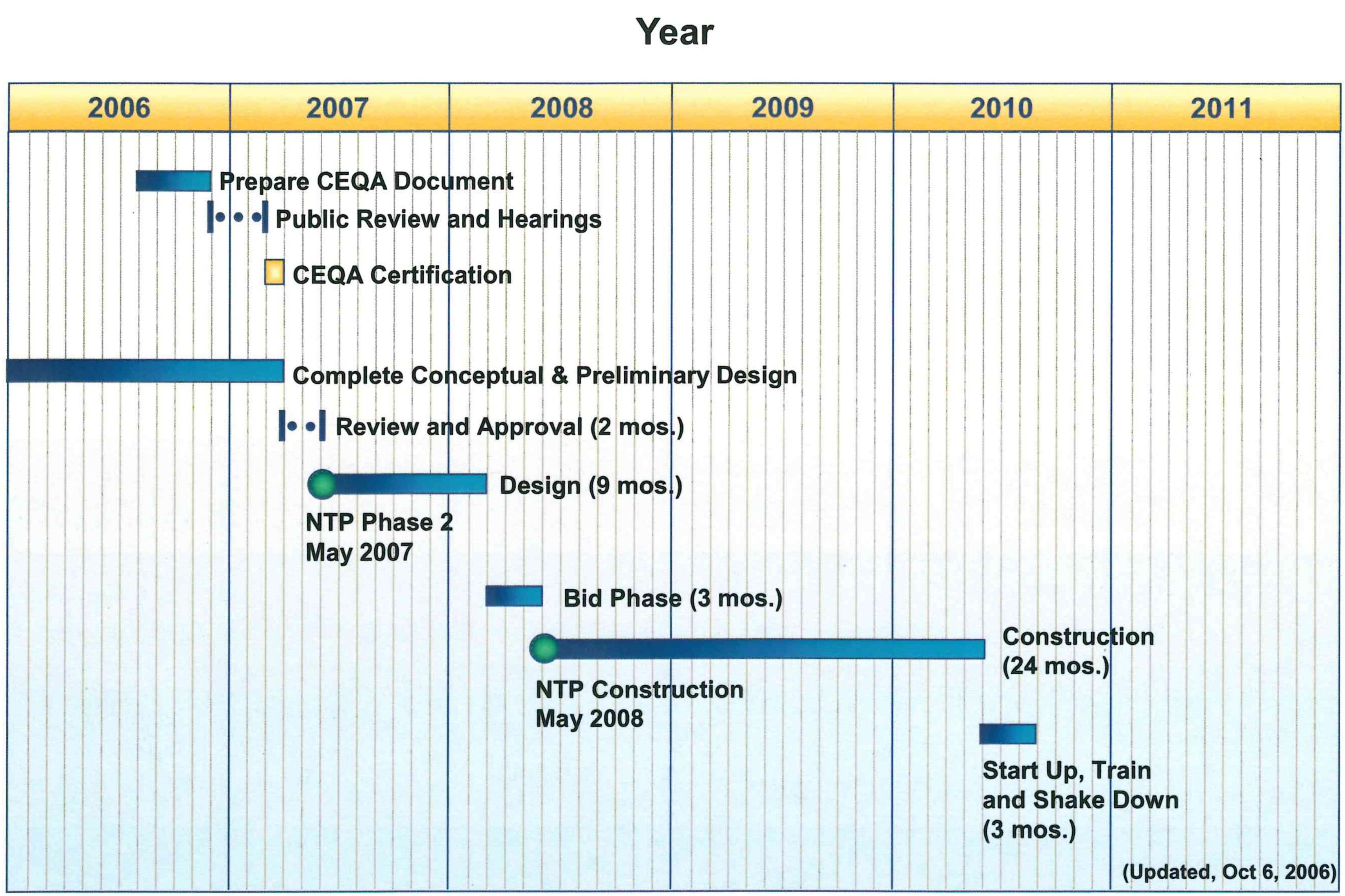

Figure 8-1
Benicia Water Reuse Project-Milestone Schedule with Zero Float

CDM

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Appendices

Appendices

BENICIA WATER REUSE PROJECT

Technical Memorandum 1 – Evaluation of Alternative Reuse Treatment Systems and Ammonia Removal Options

To: *Chris Tomasik, City of Benicia*

Cc: *PURE Members*

Date: *7 September 2004*

EXECUTIVE SUMMARY

The objective of this TM is to develop and evaluate alternative treatment approaches that will achieve the cooling water quality objectives of the Valero Refinery and that will minimize toxicity impacts to both the City's and Valero's wastewater discharges. A key issue in minimizing toxicity impacts is the effective removal and handling of ammonia from the City's wastewater. The most critical water quality criteria from a cooling water perspective relate to ammonia, silica, chloride and hardness. The water quality criteria were updated based on discussions with key Refinery staff. Using updated water quality criteria, three reuse treatment systems were developed and evaluated, as follows:

- Micro-Filtration followed by Reverse Osmosis (MF/RO) both total stream and split treatment
- MF followed by Nano-Filtration
- Granular Media Filtration followed by Electrodialysis Reversal

Based on the projected concentrations of water quality constituents in the product water from each of the three alternatives, ***it was determined that only the MF/RO treatment system could meet the cooling water quality objectives.*** A split-treatment approach, consisting of 85% RO treatment and a 15% "bypass" of filtered wastewater will meet the cooling water quality objectives and will save costs. It was further determined that ammonia removal in addition to that achieved by the MF/RO split process would be required to meet the ammonia criteria of less than 0.2 mg/L.

Alternative ammonia removal treatment systems were evaluated. The method of ammonia removal has not only cost implications but also toxicity impacts. Two ammonia removal methods were evaluated as follows:

- Ammonia removal by the reuse treatment processes
- Biological conversion of ammonia to nitrate at the City's WWTP

Ammonia removal by the MF/RO treatment system would result in an ammonia concentration of about 5 to 6 mg/L in the blended permeate. Hence, additional treatment of the permeate is required to meet the cooling water ammonia criteria of less than 0.3 mg/L. It was determined that a selective ion exchange process would be the most appropriate method to meet this final ammonia concentration criterion. Ammonia removal by the MF/RO treatment system would also create a concentrate stream with a very high concentration of ammonia in the range of 170 to 250 mg/L. This concentration would pose a serious toxicity problem for discharge with either the City's or Valero's effluent. There would also be a high concentration of ammonia in the smaller brine stream from the ion exchange process on the permeate. Thus, the ammonia from this combined concentrate and brine stream would need to be significantly reduced. The preferred alternative for accomplishing the required reduction would be a separate, 0.3 mgd biological nitrification treatment system, which would convert the ammonia to nitrate. An ion exchange system for the MF/RO permeate plus a biological nitrification treatment system for the concentrate and brine stream would have a capital cost of approximately $2.1 million. (Note: in this TM capital cost is defined as the sum of construction costs, change orders, engineering and construction management.)

Due to these high costs and the significant operation and maintenance requirements of removing ammonia from the MF/RO permeate and concentrate, ***it was determined that biological removal of ammonia at the City WWTP should be pursued.*** Hence, four options for biologically nitrifying the wastewater at the City WWTP were developed and evaluated. These included modifying the existing activated sludge (A/S) process, modifying the existing rotating biological contactors (RBC's) and combinations of both.

The estimated capital costs for these options range between $1 million and $1.8 million for the City's WWTP total build out flow of 4.1 mgd.

Based on the evaluations conducted in the development of this TM, CDM has drawn the following conclusions:

- The only technically feasible method to achieve the water quality requirements established by Valero for its cooling water is by the MF/RO treatment option.
- It is more cost-effective to remove ammonia by biological nitrification at the City's WWTP than to utilize additional treatment processes to the reuse treatment system of MF/RO.

Based on the above conclusions, CDM makes the following project recommendations:

- Adopt biological nitrification at the City's WWTP as the method of ammonia removal to meet water quality criteria. CDM will work with City's Treatment Plant staff to further refine the nitrification process selection.
- Direct CDM to begin conceptual design of the MF/RO split treatment system, using an approximate blend of 85% RO with 15% filtered wastewater.

- Direct CDM to implement the small scale pilot testing (Task 2A) using the MF/RO split treatment system.

DEVELOPMENT AND EVALUATION OF ALTERNATIVE REUSE TREATMENT PROCESSES

As part of the initial project planning in 2002, Valero evaluated the potential uses for the reclaimed water and the associated water quality criteria. Valero determined that the cooling make-up water was the most significant need, and that the reclaimed water quality mineral content should be equal to or less than the existing raw water obtained from the City of Benicia Lake Herman or North Bay Aqueduct (NBA) supplies. The critical characteristics of the existing raw water source are: no detectable ammonia, average silica concentration < 17 mg/L, conductivity less than 500 us/cm (approximately 250 mg/L TDS), chlorides less than 20 mg/L and total hardness between 50-150 mg/L.

The three membrane process alternatives identified in the CDM scope of work are shown schematically in Figure 1. CDM did a thorough evaluation of each process alternative, which included discussions with the equipment manufacturers and desk-top engineering analysis. Each alternative produces treated water with different concentrations of the critical cooling water parameters identified by Valero. Table 1 summarizes the water quality from each membrane alternative, and illustrates that the reverse osmosis process (RO) produces a permeate with a very low TDS that is less than 50 mg/L. The permeate from the RO process meets all the Valero cooing water quality criteria except that for ammonia, which can be achieved using breakpoint chlorination after the RO process. The TDS from the nanofiltration process is approximately 300 mg/L, but the concentration of chloride is greater than the criteria proposed by Valero. The EDR process produces a permeate with a TDS less than 150 mg/L, but the process doesn't reduce silica to an acceptable concentration.

As a result of the limits for chlorides and silica, CDM concluded that the only feasible reuse treatment process is MF/RO.

The TDS of the permeate from the RO process is significantly lower than Valero's current cooling water, and as a result, is very corrosive. By using a split treatment approach (i.e., routing a small percentage of the MF filtered effluent from the City's WWTP around the RO process and then blending it back with the RO permeate), the cooling water that is produced is similar in TDS to the existing supply and is less corrosive. This approach has the added benefit of reducing the capacity of the RO membrane process needed to produce 2 mgd of cooling system make-up water for the Valero Refinery.

A blend of 85% RO permeate and 15% MF filtered wastewater produces a product water with a TDS of 120 mg/L and chloride concentration of approximately 21 mg/l. Table 2 presents a comparison of the blended water quality characteristics and the Valero cooling water quality limits. This blended water has a significantly lower TDS and silica concentration than the current supply, which will reduce overall mineral build-up in cooling water system.

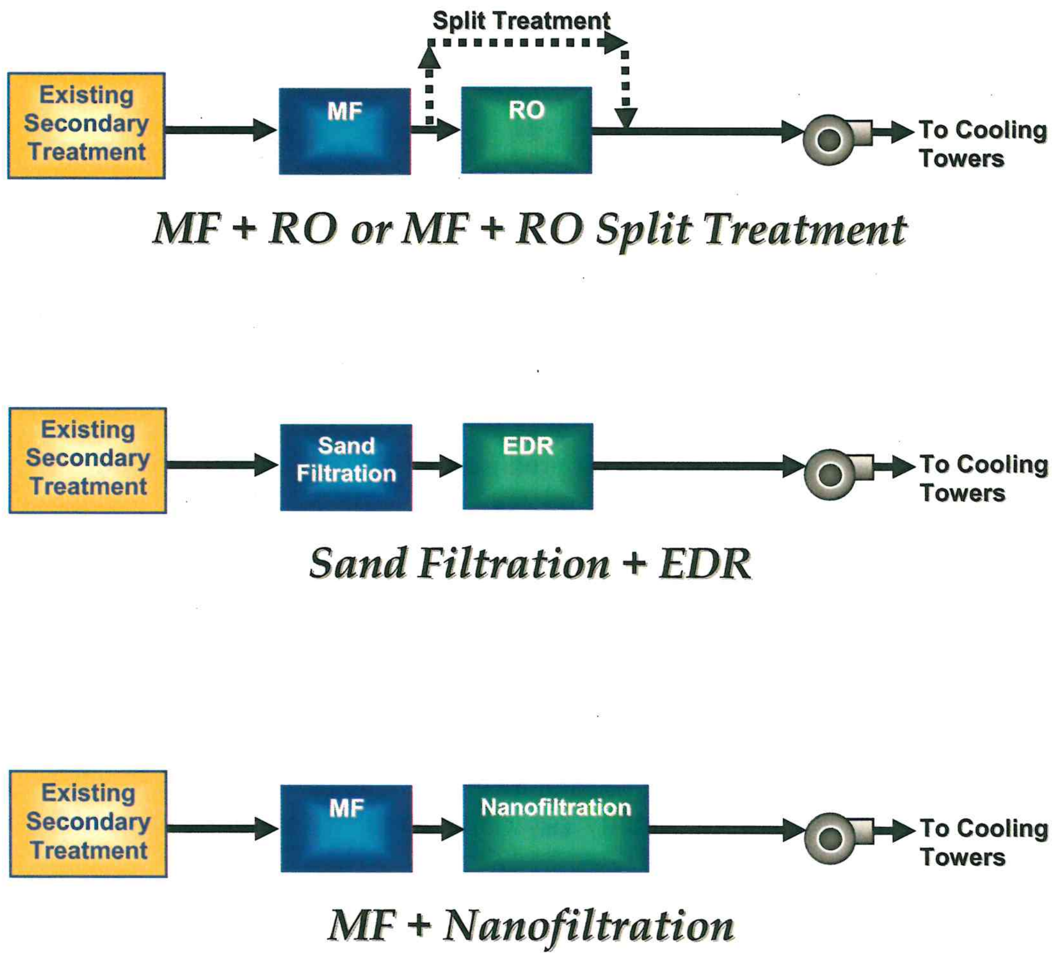


Figure 1 – Membrane Process Alternatives

Table 1 Treated Water Comparison					
Water Quality Parameter	***Assumed WWTP Effluent Water Quality (1) mg/L***	***Valero Water Quality Limits (3)***	***Microfiltration Reverse Osmosis Permeate mg/L***	***Microfiltration Nanofiltration Permeate mg/L***	***Electro-Dialysis Reversal mg/L***
Cations					
– calcium	25		0.5	4	0.8
– magnesium	18		0.3	3	0.2
– sodium	130		10	80	7
– potassium	18		2	14	0.2
– ammonia (2)	3	<0.2	***0.3(4)***	***2.3***	***0.3***
– barium	0.1		<0.01	0.03	<0.01
– strontium	1		0.02	0.2	0.02
– aluminum	0.1	1	<0.01	0.05	
– copper	0.03	0.05	<0.005	0.01	0.01
Anions					
– bicarbonate	190	104	11	70	45
– sulfate	90		1	11	20
– chlorides	120	20	4	***100***	18
– phosphate	3.0	3	<0.2	0.5	
– fluoride	1		0.1	1	0.2
– nitrate (2)	25		6	2.1	2.3
– silica	22	17	1	10.3	***22***
–					
TDS	650	250	30	***300***	116
Tot. Hardness	130	<200	5	22	3
General					
– pH	7.0	6-8	***5.9***	6.6	6.3
– Langlier I	-0.9				-3.71
– BOD	17		<1	<2	<5

(1) Data were obtained from Table 2 in the October 1, 2002 memorandum prepared by EOA, Inc. for the City of Benicia labeled *Task 1 – Confirm Recycled Water Use Potential and Water Quality Requirements*
(2) Assumed that the ammonia concentration in the wastewater will be reduced from 25 mg/L to 3 mg/L by nitrification, so that the ammonia concentration would be relatively low in the permeate. Without nitrification the ammonia concentration in the permeate would be 2-3 mg/l for the RO and EDR alternatives and 18 mg/l for nanofiltration.
(3) Based on results of meeting of August 31, 2004 between Steven Penney of Valero and Doug Brown of CDM.
(4) Will require further reduction to meet ammonia criterion as shown in Table 2

The blended water will also be low in hardness (<50 mg/L), which is lower than desired by Valero for corrosion control, so lime may be added to prevent corrosion. Therefore, it is desirable to increase the amount of MF filtered wastewater to be blended with the permeate, if it is found that the chloride concentration of the wastewater is actually less than the value shown (120 mg/L) in Tables 1 and 2.

Table 2 Blended RO Permeate Water Quality					
Parameter	***Units***	***Benicia Effluent Water Quality***	***RO Permeate Water Quality***	***Blended Water Quality @ 85% Permeate***	***Valero Cooling Water Quality Limits***
calcium	mg/L	25	0.5	4	
magnesium	mg/L	18	0.3	3	
sodium	mg/L	130	10	27	
potassium	mg/L	18	2	4	
ammonia	mg/L	1	0.3	***0.4***(1)	***<0.2***
bicarbonate	mg/L	190	11	37	104
sulfate	mg/L	90	1	14	
chloride	mg/L	120	4	21	20
phospahate	mg/L	2	0.2	0.5	3
fluoride	mg/L	1	0.1	0.2	
nitrate	mg/L	25	6	9	
silica	Mg/L	22	0.7	4	17
Hardness	Mg/L	130	5	23	<200
TDS	Mg/L	650	30	120	250

(1) Ammonia in the blended product water would be eliminated using breakpoint chlorination.

AMMONIA REDUCTION ALTERNATIVES

One of the critical water quality requirements for the Valero Refinery cooling water supply is elimination of the ammonia that is present in the treated wastewater from the Benicia WWTP. The two basic approaches are either to eliminate the ammonia as part of the wastewater treatment process or to use the MF/RO reclamation treatment process to remove the ammonia. If the wastewater treatment plant is operated in the nitrification mode, the 20-27 mg/L of ammonia will be converted to nitrates, and the concentration of the ammonia in the feed water to the reuse treatment process will be approximately 1 mg/L. As been previously described, the RO treatment process will reject the ammonia and reduce the concentration of ammonia in the permeate to less than 0.2 mg/L.

The proposed split stream reclamation treatment process will blend the RO permeate with a small percentage (10-15%) of filtered wastewater (split stream) to stabilize the permeate and reduce the corrosiveness of the reclaimed water. The ammonia concentration in the blended water will be approximately 0.4 mg/L, which can be removed by breakpoint chlorination. The ammonia in the feed water to the RO process will be concentrated in the reject stream, and will be approximately 6-7 mg/l if the RO system is operated at 85% recovery. Generally this concentration of ammonia does not exhibit any toxic effects at the estimated pH for the concentrate, and it is proposed to combine the RO concentrate with the remaining secondary effluent that will be discharged from the wastewater treatment plant.

If the WWTP is not operated in the nitrification mode, the high ammonia concentration in the feed water to the MF/RO reclamation treatment process will result in high ammonia concentrations in the blended permeate and the reject stream from the RO process. As a result of the 20-27 mg/L ammonia concentration in the feed water, the permeate from the RO process will have 2-3 mg/L of ammonia, and when the RO permeate is blended with the split stream the resulting ammonia concentration in the blended flow will be 5-6 mg/L. The concentration of ammonia in the reject stream from the RO process will also increase significantly to 170 to 250 mg/L for recovery ratios of 85% to 90%, respectively. It will be necessary to provide additional treatment for both the blended permeate and the concentrate to reduce the ammonia concentration to acceptable concentrations.

Treatment Options for Removing Ammonia from Blended Permeate

The potential treatment options for reducing the ammonia in the blended permeate are breakpoint chlorination, ion exchange and air stripping. Breakpoint chlorination has the lowest capital cost and operating cost. It will, however require high doses of sodium hypochlorite to reduce the ammonia, but this will have the adverse impact of increasing the chloride concentration, another critical water quality parameter. Ion exchange using a sodium based zeolite is also an effective and common process, and if properly operated should have minimal impact on the chloride concentration in the blended permeate. The regeneration process for the ion exchange system will generate a concentrated solution of sodium chloride and ammonium chloride that must be treated at the treatment plant or discharged. The additional TDS and ammonia load on the WWTP will have an adverse impact on operation of the WWTP, so it is anticipated that the waste from the ion exchange system will have to be combined with the RO concentrate for separate treatment.

Air stripping will require raising the pH of the blended flow to between 10-11 to convert the ammonium ion to ammonia gas that can be stripped. After passing through air stripper the pH will have to be lowered to pH 8 to reduce the scale forming potential of the water. The air stripping process will remove approximately 30,000 lbs of ammonia per year from the blended flow, and it is also expected that due to air quality requirements and odor potential, it will be necessary to remove ammonia from the air stream prior to discharge to the atmosphere. This will create another liquid ammonia waste stream that must be returned to the WWTP for subsequent treatment.

Because of the adverse impacts associated with the breakpoint chlorination and air stripping alternatives, ion exchange is considered the only alternative suitable for removing ammonia from the blended permeate. The capital cost for a 2 mgd treatment system is approximately $600,000 including equipment, installation, engineering and contingency. This does not include the cost for treating the ion exchange regeneration brine.

Treatment Options for Removing Ammonia from RO Concentrate

A 2 mgd RO process will generate 200,000 -350,000 gallons of concentrate per day when operating at 90-85% recovery. The concentrate will have 4,000 to 5000 mg/L TDS, 170 mg/L-250 mg/L of ammonia and 10 -20 mg/L of BOD. There will also be the 40,000 gpd brine stream from the above described ion exchange process on the RO permeate stream. The ammonia in the combined RO concentrate and ion exchange brine must be reduced to less than 20 mg/L, so it can be discharged downstream of the WWTP. The treatment options to do this are a separate biological nitrification system, ion exchange, or air stripping.

There are two problems associated an ion exchange system. The first is the high concentrate of calcium and magnesium in the concentrate will be preferentially adsorbed resulting in very frequent regeneration requirements. The second problem is there is still 10,000 -20,000 gpd of a high ammonia brine to discharge or treat.

There are also significant problems associated with the air stripping alternative. When the pH of the concentrate is raised to convert the ammonium to gas, a large percentage of the minerals in the concentrate will precipitate. This will require a large reactor clarifier and generate 10,000 to 15,000 gpd of sludge (2000 -3000 lbs/day of dry solids), which must be dewatered and landfilled. The second problem associated with this alternative is treatment of the ammonia laden air discharged from the air stripper. It is estimated that there will be 500 lbs/day of ammonia stripped from the RO concentrate that must be subsequently scrubbed from the air stripper discharge. Treatment options will depend on the concentration of ammonia in the air scrubber discharge.

Because the problems associated with ion exchange and air stripping, the only reasonable alternative for treatment of the RO concentrate is separate biological treatment. A separate nitrification treatment process to treat the RO concentrate and ion exchange regeneration brine will require aeration basins, clarifiers and potentially storage tanks for a carbonaceous food source. The sludge treatment, disinfection and other equipment at the existing treatment plant could be used for the ancillary processes. The estimated capital cost for a separate 400,000 gpd biological treatment system is approximately $1,500,000.

In summary, there are high capital costs associated with eliminating ammonia in the blended permeate and reducing the ammonia in the RO concentrate, if the feed water from the WWTP is not fully nitrified to reduce ammonia to less than 2 mg/L. The estimated capital cost to treat both streams is greater than $2.1 million, and there are significant operational costs and labor requirements associated with the ancillary treatment systems.

ALTERNATIVE BIOLOGICAL PROCESS MODIFICATIONS AND ADDITIONS AT CITY'S WWTP FOR AMMONIA REMOVAL

The advantage of the biological approach to ammonia removal is that it is converted to the nitrate form and not concentrated in a reject or brine stream. Eliminating ammonia altogether will help to minimize toxicity at either outfall as the byproducts of nitrification are non-toxic.

Design Criteria

CDM evaluated existing and build-out conditions for the various alternatives to modify the Benicia WWTP to a nitrification plant. Figure 2 shows the overall plant liquid stream schematic after completion of the I/I Improvement Project - WWTP Wet Weather Improvements, currently under construction. Note that the flows shown on Figure 2 are neither current flows nor projected build-out flows. They were taken from the design drawings for the current WWTP project under construction. The analysis presented in this TM looks at both current flows and project build-out flows. Table 3 provides a summary of the flow and primary effluent design criteria used in this analysis. Flow and loading criteria are representative of maximum month values for the respective seasons based on actual WWTP data for the past three years. Primary effluent concentrations are similar to the design assumptions developed for the 2003 WWTP I/I Improvement Project. These assumptions are similar to what is currently produced and account for recycle and side stream flows. Side stream flow is estimated to be 0.3 mgd year-round. The build-out maximum month flow is equal to the design maximum month flow in the latest upgrade project (4.2 mgd + 0.3 mgd side streams).

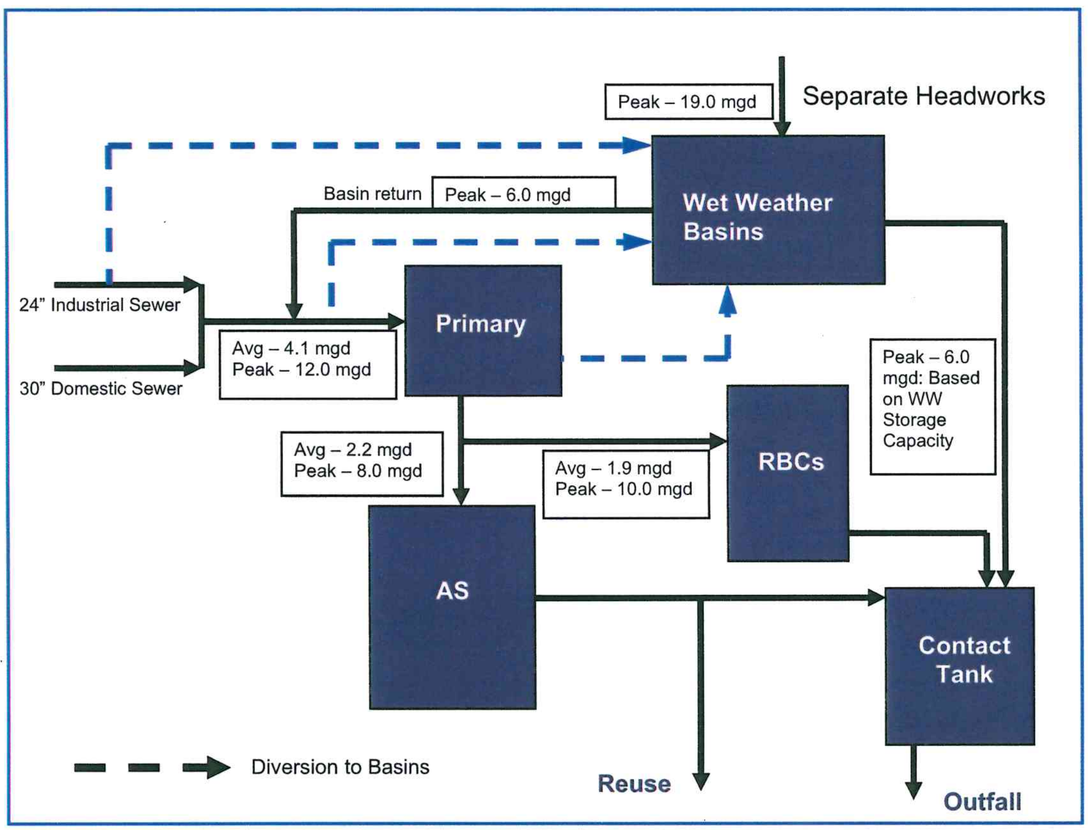


Figure 2 - Overall Plant Schematic

Table 3
Design Criteria for Nitrification Evaluation[1]
(Maximum Month, 30-day averages)

	Current		*Build-out*	
Parameter	Winter	Summer	Winter	Summer
Flow, mgd	3.7	3.3	4.5	4
Primary Effluent				
BOD5, ppd	4,500	4,000	5,500	4,900
BOD5, mg/L	150	145	150	150
TSS, ppd	3,800	2,700	4600	3,300
TSS, mg/L	120	100	120	100
TKN, ppd	680	600	825	735
TKN, mg/L	22	22	22	22
NH3, ppd	460	330	560	500
NH3, mg/l	15	15	15	15
Temp, deg. C	17	26	17	26

[1]Based on 2003 and 2004 plant data

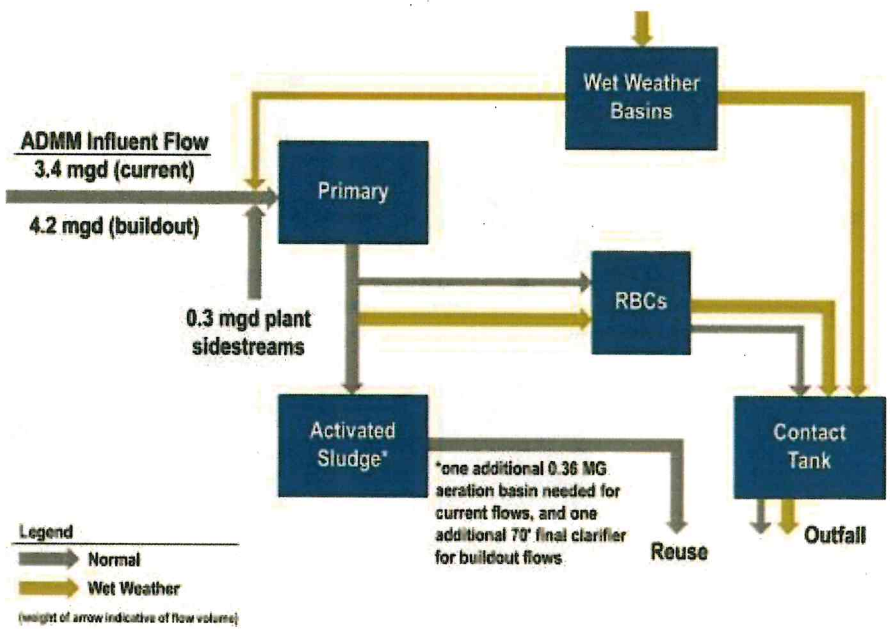
This evaluation considered full nitrification (effluent ammonia less than 1 mg/L) as the effluent criteria. Achieving this degree of nitrification will result in effluent 5-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) of less than 10 mg/L.

Process Options

CDM investigated four different process flow schemes to convert the existing WWTP into a nitrification plant:

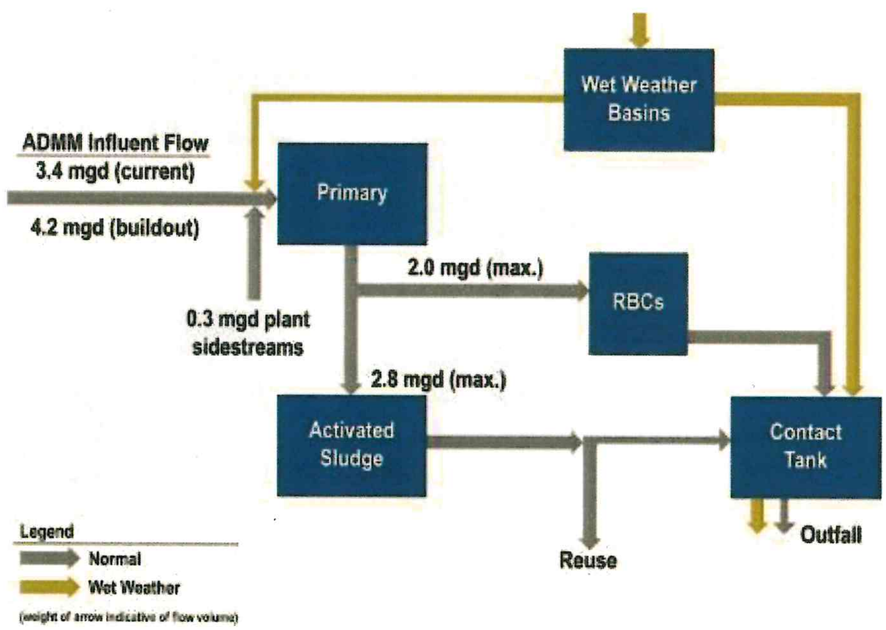
- **Option 1 Activated sludge** – Expanding the activated sludge (A/S) process to allow increasing solids retention time (SRT) to achieve nitrification and reserving the rotating biological contactors (RBCs) for wet weather treatment only
- **Option 2 Split-flow** – Operating the RBCs and A/S processes in parallel, with both processes achieving full nitrification
- **Option 3 RBC Roughing Process** – Using the RBCs for pretreatment ahead of the A/S process
- **Option 4 RBC Nitrification** – Operating the A/S process in a partial nitrification mode, and using the RBCs to complete the nitrification process

Figures 3 through 6 present schematics for these alternatives.



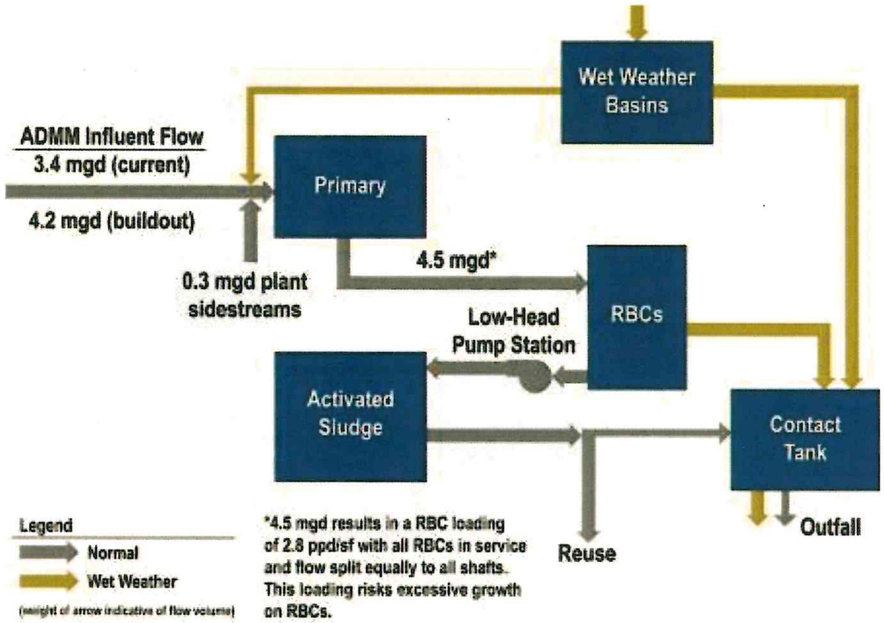
Note: RBCs Held standby for Peaks (Activated Sludge Sized for Nitrification)

Figure 3 - Option 1 - Activated Sludge Process



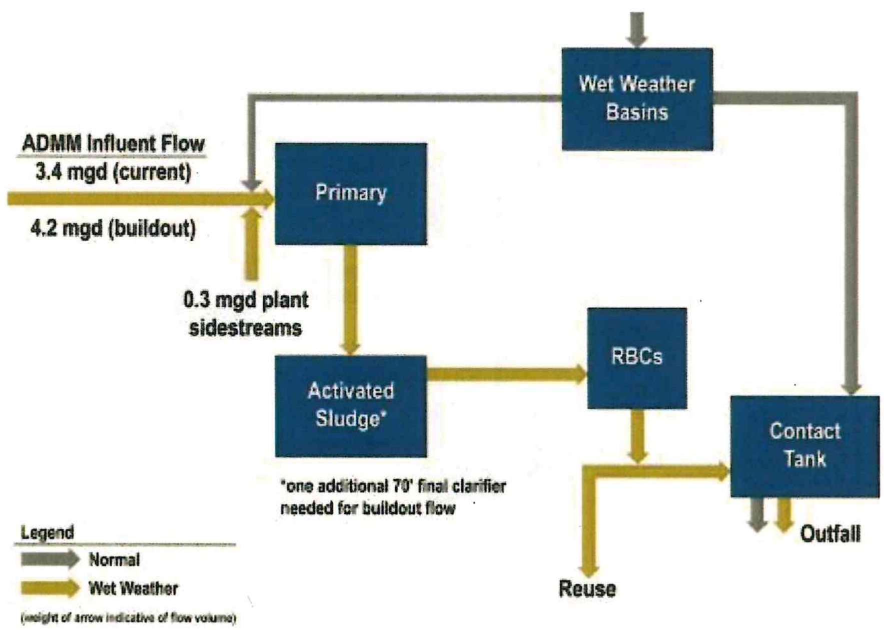
Note: Split Flow Between RBCs and Activated Sludge for Full Nitrification During Dry Weather

Figure 4 - Option 2 – Split Flow



Note: RBCs Used for Roughing Ahead of Activated Sludge

Figure 5 - Option 3 – RBC Roughing Process



Note: RBCs Used for Nitrification Following Activated Sludge

Figure 6 - Option 4 – RBC Nitrification

Option 1 - Activated Sludge Process

The controlling design parameter for the A/S process is SRT. At 17 degrees, and using a nitrification safety factor of two (twice the minimum SRT needed for nitrification), the design A/S process SRT for all evaluations is nine days.

As seen in Table 3, the design for current loadings result in a maximum month BOD loading of 4,500 lbs. per day. The existing aeration basins are designed for a maximum BOD loading of 4,130 lbs. per day. Therefore, for build-out condition an expansion of the A/S system is required. A third aeration basin equal in size to the existing two basins was assumed. Even with this expansion, MLSS concentration at the build-out condition (4,400 mg/L) is reaching the upper limits of conventional design values. Also, a third final clarifier and an additional blower (allowing for one redundant unit) would be required to treat build-out flow.

Table 4 lists the key design criteria for the A/S plant option. Evaluations were performed for current and build-out conditions.

Table 4
Option 1 – Activated Sludge

Parameter	***Current***	***Build-out***
Aeration Basins (0.36 mgd each)	3	3
Final clarifiers (70-ft diameter)	2	3
SRT, days	9	9
MLSS, mg/L	3,700	4,400
Final clarifier solids loading, $lbs/d/ft^2$	25	25
RAS rate, % influent flow	70	70
No. of 1500 icfm blowers required: Currently 3 blowers installed with 1 as a standby	2.1	2.5

Option 2 - Split Flow

The existing RBCs are currently organically and hydraulically under-loaded. The Split Flow alternative takes advantage of the existing RBC capacity by increasing the flow rate to the RBCs, thereby taking sufficient loading off the A/S process to allow it to become a nitrification process without expansion.

Determination of the nitrification capacity of the RBCs is critical to this evaluation because any primary effluent routed to the RBCs must be fully nitrified; otherwise it cannot be combined with the A/S effluent for reuse. BOD and ammonia loading curves, commonly used in the wastewater industry for sizing of RBC's, were utilized to estimate their BOD and ammonia removal capacities. (Rating curves were provided by Envirex and are on-file in CDM's project files.)

Based on the design parameters in Table 3 above and referenced loading curves, CDM estimates that the existing RBC process should be able to nitrify approximately 1.7 mgd with resultant ammonia concentration of 1 mg/L. Therefore, the combination of the RBCs and the existing A/S process are theoretically capable of fully nitrifying all flows up to the build-out flow condition. This needs verification with City's WWTP staff based on actual plant operating conditions and experience.

Table 5 lists the key design criteria for the split flow option. Due to the superior quality of A/S plant effluent, the amount of flow sent to the A/S portion of the plant should be maximized versus the amount sent to the RBC's. On a preliminary basis, CDM estimates that the amount of flow that can be nitrified by the A/S system is in the range of 2.5 to 2.8 mgd based on allowable solids loading to the final clarifiers. Maximizing A/S plant flow also allows more wet-weather flow to be routed through the RBCs. This is preferable to taking peak flows through the A/S process, which may cause clarifier washout of the light A/S biomass. The RBC final clarifiers are better suited to handle peak flows because of the excellent settling characteristics and low concentration of the RBC solids. The hydraulic capability of the plant to accommodate such flow splits needs further evaluations.

Table 5
Option 2 – Split Flow

Parameter	*Current*	*Build-out*
Flow to A/S process, ADMM, mgd	2.8	2.8
Flow to RBC process, ADMM, mgd	0.9	1.7
SRT, days (A/S plant)	9	9
MLSS, mg/L (A/S plant)	4,500	4,500
Final clarifier solids loading (lbs/d/ft^2) (A/S plant)	25	25
RAS rate (A/S plant), percent of influent flow	70	70
No. of 1500 icfm blowers required: Currently for A/S plant 3 installed with 1 as a standby	1.5	1.5

Option 3 – RBC Roughing Process

This option involves the use of the RBC process for removing the majority of the BOD, thereby allowing the existing A/S process to easily convert to nitrification. If all flow and load are being directed to the RBCs, it will be necessary to re-stage the process to prevent overgrowth on the RBC discs. This evaluation assumes all flow is split equally to the first five RBCs in the train. The sixth RBC is a high-density RBC, and it is not recommended that it be made a part of the first stage. This results in acceptable loading rates for the current design condition, but risks RBC overload in the build-out design condition. The maximum recommended first stage BOD loading is 2.5 gpd/sf (Figure 7) and the loadings exceed this in the build-out condition. As flows increase and the RBCs start become overloaded, more flow could be directed to the A/S process (making this a variation of the split flow process).

Table 6 lists the key design criteria for the RBC roughing option.

Table 6 Option 3 – RBC Roughing		
Parameter	**Current**	**Build-out**
First stage RBC hydraulic loading (gpd/ft^2 – 2.5 max.)	2.3	2.8
BOD/TSS/TKN to A/S Process (mg/L)	25/25/10	30/30/15
SRT, days (A/S process)	13	13
MLSS, mg/L (A/S process)	1,600	2,500
Final clarifier solids loading (lbs/d/ft^2) (A/S process)	11	21
RAS rate (A/S process), % of influent flow	70	70
No. of 1500 icfm blowers required – A/S process(3 currently installed)	0.7	1.1

Option 4 - RBC Nitrification

This option was recommended by the RBC manufacturer and appears to be a feasible alternative. In this option, instead of expanding the current A/S process, it would be operated in a mode to achieve partial nitrification. This option would require process air supply expansion to maintain a standby blower and an additional final clarifier to treat the build-out flows. A detailed review of the plant hydraulics may lead to the need for a pump station to deliver flow from the A/S secondary clarifiers to the RBCs. Table 7 lists the key design criteria for the RBC nitrification option.

Table 7 Option 4 – RBC Nitrification		
Parameter	**Current**	**Build-out**
A/S plant effluent soluble ammonia, max. (mg/L)	15	12
RBC loading rate, gpd/ft^2	1.9	2.3
SRT, days (A/S plant)	5	5
MLSS, mg/L (A/S plant)	3,400	4,100
Final clarifier solids loading (lbs/d/ft^2)	23	23
Final clarifiers (70-ft diameter)	2	3
RAS rate, % of influent flow	70	70
No. of 1500 icfm blowers required (3 currently installed)	1.9	2.4

Based on the preliminary analysis of biological ammonia removal options, Table 8 contains a summary of the additional facilities and estimated capital costs that would be needed to implement each ammonia removal option.

Table 8 Major Additional WWTP Facilities and Estimated Capital Costs				
	Option			
	1 – Activated Sludge	**2 – Split Flow**	**3 – RBC Roughing**	**4 – RBC Nitrification**
Items needed	• New aeration basin with odor control • One additional blower	• No major new facilities	• RBC Flow Distribution • 4 mgd pump station	• One additional blower • New secondary clarifier
Estimated Costs Construction Capital	 $1,300,000 $1,800,000	 <$200,000	 $750,000 $1,000,000	 $800,000 $1,100,000

Summary Evaluation of Alternative Biological Ammonia Removal Alternatives

As can be seen from the preliminary costs presented in Table 8, additions and modifications to the City WWTP to provide complete nitrification are estimated to cost in the range from less than $0.2 million to $2 million dollars, depending on the amount of wastewater to receive full nitrification. Prior to selecting to best ammonia removal option, several factors require further evaluation including potential impacts to the down stream micro-filtration process resulting from any differences in nitrified effluent characteristics, constant flow rate of recycled water to be produced, WWTP detailed hydraulics, operating experience relating to the RBC's and accommodation of wet weather flows. CDM intends to discuss these factors with the plant staff and conduct further evaluations before coming to a joint recommendations on the best option to pursue.

Conclusions

Based on the evaluations conducted in the development of this TM, CDM has drawn the following conclusions:

- The only technically method to achieve the water quality requirements established by Valero for its cooling water is by the MF/RO treatment option.

- It is more cost-effective to remove ammonia by biological nitrification at the City's WWTP than to utilize additional unit processes to remove ammonia from the RO permeate and brine.

Recommendations

Based on the above conclusions, CDM makes the following recommendations:

- Adopt biological nitrification at the City's WWTP as the method of ammonia removal to meet water quality criteria. CDM will work with City's Treatment Plant staff to further refine the nitrification process selection.

- Direct CDM to begin conceptual design of the MF/RO split treatment system, using an approximate blend of 85% RO with 15% filtered wastewater.

- Direct CDM to implement the small scale pilot testing (Task 2A) using the MF/RO split treatment system.

City of Benicia – Water Reuse Project
Draft Supplement to Technical Memorandum No. 1 – Biological Nitrification Alternatives

TO: Chris Tomasik

CC: PURE Members

DATE: 30 November 2005

Executive Summary

Development of Biological Nitrification Treatment Alternatives

The purpose of this Supplement is to identify and screen potentially available biological nitrification technologies in order to determine the feasible options worthy of further evaluation. In order to provide additional assurance that the best nitrogen control technology is selected, 11 biological treatment technologies that would potentially provide full-time nitrification were identified and screened. Six biological nitrification technologies were selected for further analysis from the technologies found most feasible to meet project objectives. Conceptual designs for six alternatives were prepared and analyzed for performance, reliability and cost-effectiveness. Three alternatives involve extensive modifications to the City's existing WWTP. They require that the entire secondary treatment system be included in the process development, along with accommodations for wet weather operations. Three other alternatives are basically stand alone systems, which can be sized solely to meet the flow demands of the Water Reuse Project. The six alternatives analyzed are described in **Table ES-1.**

Table ES-1
Biological Nitrification Alternatives for Ammonia Removal

Alternative	***Description***
1	Expand existing activated sludge system – use 2 existing aeration Basins (AB's) add a 3rd secondary clarifier (SC), 3rd return activated sludge (RAS) pump and 3 process air blowers. *Nitrifying Activated Sludge (2 AB's & 3 SC's)*
2	Expand existing activated sludge system – add 3rd AB, 3rd SC, 3rd RAS Pump and 3 blowers. *Nitrifying Activated Sludge (3 AB's & 3 SC's)*
3	Convert primaries to chemically enhanced primary treatment (CEPT) - add a 3rd secondary clarifier, 3rd RAS pump, 3 process blowers and chemical feeding system. *Nitrifying Activated Sludge & CEPT*
4	Add stand-alone tertiary nitrifying biological aerated filters. *Nitrifying BAF's*
5	Add stand-alone tertiary submerged, fixed-film nitrification system. *TSFF Nitrification*
6	Add stand-alone tertiary nitrifying trickling filters. *NTF's*

Overview Nitrifying Activated Sludge Alternatives (Alternative Nos. 1, 2 & 3)

Of the three NAS alternatives, Alternative No. 2 provides the highest degree of reliability because nitrification can be maintained and the required wet weather flows can be passed with either one AB or one SC out of service. Alternative No. 3 provides less reliability than Alternative No. 2 because nitrification will likely be lost when one AB is removed from service; however, the activated sludge process can still pass the required wet weather flow with one SC out of service. Of the three full plant nitrifying activated sludge processes, Alternative No. 1 provides the least amount of reliability because loss of an AB will stop nitrification and loss of a SC will prevent the SCs from passing the required wet weather flow. **Table ES-2** presents a summary of the flow rates that each of these three alternatives can handle and still reliably meet the secondary effluent ammonia limit of 2 mg/L.

Table ES-2
Summary of Reliable Flow Limitations for Nitrifying Activated Sludge Alternatives

Alternative	*Average Day Max Month, mgd*	*Peak Hourly Flow, mgd*
Estimated Flow at Build Out	4.5	8
Current Flow	3.7	8
Alt No. 1 – NAS with 2 AB's & 3 SC's	2.0 to 3.2	5.4 to 8.4
Alt No. 2 – NAS with 3 AB's & 3 SC's	3.2 to 4.0	8.3 to 10.8
Alt No. 3 – NAS & CEPT with 2 AB's & 3 SC's	2.4 to 3.6	6.4 to 9.9

Overview of Stand-Alone Biological Nitrification Systems

Alternative No. 4 Biological Aerated Filters

Biological Aerated Filters (BAF's) are a type of attached growth biological treatment process that are used for tertiary nitrification. Nitrifying bacteria grow on the surface of the media and convert the ammonia to nitrate. BAF's have characteristics of both activated sludge systems and trickling filters. Mechanically, they function similar to a water filter in that they must be backwashed periodically. Hence, there is backwash wastewater that must be recycled back to the main plant head works. The system has backwash pumps, process air blowers and backwash air blowers. BAF's have a high profile of approximately 25 ft in height.

Alternative No. 5 Tertiary Submerged Fixed-Film Reactor Systems

Tertiary submerged fixed-film reactor systems are composed of a reaction vessel and either fixed or moving-bed media on which nitrifying bacteria grow. Air is diffused into the water-media culture much like a typical AS aeration basin. Fixed media consist of either ropes that are attached to frames, or plastic crates, similar to those used in packed bio-towers. Moving-bed media are made of either sponges or small plastic elements. Since maintenance of the fixed media has presented challenges at some installations, only plastic media of the moving-bed type were considered. TSFF systems have low profiles, are similar to aeration basins and would project about five feet above grade.

Alternative No. 6 Tertiary Nitrifying Trickling Filters

Nitrifying trickling filters (NTF) are attached growth biological treatment processes that allow the nitrifying bacteria to grow on the surface of solid media, as the wastewater flows over the media. This is opposite of the suspended growth processes (i.e., NAS, as in Alternative Nos. 1, 2 & 3 and TSFF systems, as in Alternative No. 5) where the bacteria are "suspended" in the wastewater. The NTF's units for Benicia would be approximately 42-ft in diameter and 15-ft high.

Schematic Diagrams and Conceptual Site Plans

To aid the reader's understanding of the six alternatives, schematic diagrams and conceptual site plans are presented in Sections 3 and 4 of the Supplement.

Estimated Construction Costs of Biological Nitrification Alternatives

Conceptual designs were developed and construction cost estimates were prepared for each of the six alternatives. For the three stand-alone alternatives, Alternative Nos. 4, 5 and 6, manufacturers were contacted for budgetary estimates for the respective equipment. Unit prices for various components and surcharges for electrical and instrumentation and control systems were used that are similar to those used in the other TM's. The construction estimates indicate that Alternative No. 4 BAF's has the highest estimated cost at approximately $3.67 million, while Alternative No. 1 NAS (2 AB's & 3 SC's) has the lowest estimated cost at approximately $1.79 million. However, Alternative No. 1 has reliability limitations, as noted above. Alternative No. 6 Nitrifying Trickling Filters has the second lowest estimated construction cost at $2.06 million.

Estimated Operating & Maintenance Costs of Biological Nitrification Alternatives

Operating requirements, including power, labor, chemicals and other consumables were estimated for each of the six alternatives. Power was estimated at $0.12 per kilowatt hour (kWhr); labor at $50 per hour, including City administrative overhead. Chemical costs used were current local market rates. For Alternative Nos. 1, 2 and 3, which are dependent on the total flow to the entire WWTP, an annual average flow over the 20-year planning period was assumed at 3.8 mgd. For Alternative Nos. 4, 5 and 6, a constant flow of 2.55 mgd (as the required input to the MF/RO system) over the 20-year period was assumed. Alternative No. 3 Nitrifying Activated Sludge with Chemically Enhanced Primary Treatment has the highest estimated operating cost at approximately $314,000 per year. Alternative No. 6 NTFs has the lowest estimated operating cost at approximately $165,000 per year. The estimated operating cost of the other four alternatives ranged between $192,000 and $242,000 per year.

Quantitative Evaluation of Alternatives

The capital cost of a project includes both the initial construction cost plus engineering and construction management costs, required to implement the project. An amount of 35% of the estimated construction cost has been added to account for these costs.

The capital and annual O&M cost estimates presented herein are for comparative purposes only. These cost estimates are used to determine the biological nitrification alternative that is the most cost-effective in relation to the other alternatives. Using estimated capital and annual O&M costs for each alternative system, present worth values were developed to compare the life-cycle costs of the six alternatives. Present worth is defined as that amount of money it takes to fund the capital investment of a project, as well as its annual operating and maintenance costs, over a period of time, given the cost of money (interest) during the evaluation period. For this analysis, the time period used was 20 years and the interest rate was six percent. **Table ES-3** presents the results of this analysis.

Table ES-3
Summary of Economic Analysis of Biological Nitrification Alternatives

Component	*Alt No. 1 NAS (2&3) $1,000's*	*Alt No. 2 NAS (3&3) $1,000's*	*Alt No. 3 NAS&CEPT $1,000's*	*Alt No. 4 BAFs $1,000's*	*Alt No. 5 TSFF $1,000's*	*Alt No. 6 TNTF $1,000's*
Estimated Construction Costs (1)	$1,790	$3,310	$2,340	$3,670	$2,880	$2,060
Add 35% for Engineering and CM	$630	$1,160	$820	$1,280	$1,010	$720
Total Estimated Capital Cost	$2,420	$4,470	$3,160	$4,950	$3,890	$2,780
Estimated Annual O&M Costs (2)	$202	$211	$314	$242	$192	$165
Present Worth of O&M Costs (3)	$2,320	$2,420	$3,610	$2,780	$2,200	$1,890
Total Estimated Present Worth Values	**$4,740**	**$6,890**	**$6,770**	**$7,730**	**$6,090**	**$4,670**

(1) From Tables 5-1 through 5-6
(2) From Tables 6-2, 6-4, 6-6, 6-8, 6-10 & 6-12
(3) PWF: i = 6% and n = 20 yrs

As can be seen from inspection of **Table ES-3**, Alternative No. 6 has the lowest present worth value among the six alternatives analyzed. Alternative No. 1 has the next lowest present worth value by approximately 1.5%. Although Alternative No. 1 has the lowest estimated capital cost, it has significant reliability limitations in that it cannot consistently meet the maximum secondary effluent ammonia criterion of 2 mg/L.

Qualitative Evaluation of Alternatives

In addition to capital cost, operating costs and overall present worth values, it is appropriate to evaluate other qualitative factors to aid in the selection of the best biological nitrification process. **Table ES-4** contains a tabular summary of pertinent qualitative factors and an assessment of how each alternative compares to each factor.

Table ES-4
Summary of Qualitative Evaluation of Biological Nitrification Alternatives

Qualitative Factors	Alt No. 1	Alt No. 2	Alt No. 3	Alt No. 4	Alt No. 5	Alt No. 6
Impact on Existing Facilities	Moderate	High	Moderate	Low	Low	Low
Ease of Operation	Good	Good	Moderate	Moderate	Good	Good
Ease of Implementation	Moderate	Difficult	Moderate	Good	Good	Good
Incrementally Expandable	Difficult	Difficult	Difficult	Moderate	Moderate	Moderate
Equipment Reliability	Good	Good	Good	Good	Good	Good
Process Reliability	Limited	Good	Limited	Good	Limited	Good
Proven Technology	Good	Good	Good	Good	Limited	Good
Process Complexity	Moderate	Moderate	Moderate	High	Low	Moderate
Power Demand	High	High	High	Moderate	Low	Lowest
Visual Impact	Low	Low	Low	High	Low	Moderate

Constructing additional process units to expand the existing biological treatment system will be disruptive to the City's WWTP, whereas a stand-alone system will not disrupt plant operations. All of the alternatives are relatively easy to operate, although the chemical addition system for Alternative No. 3 and the BAF backwashing system for Alternative No. 4 will require more operator attention.

It may be decided to stage the Water Reuse Project, that is build it in stages, say from an initial capacity of 1 mgd to 2 mgd. Disruption to the existing plant operations would be relatively high and similar to the impacts, if the full, 2 mgd system were built. The stand-alone system can be staged with moderate impacts for the second stage of development.

Process reliability and technology for NAS alternatives are well proven and understood. Extensive operating performance data exist for plants operating in the NAS mode. Adequate operating data for nitrifying BAF's are also readily available, although less extensive than NAS systems. Although CDM is comfortable that the nitrification processes of Alternatives 5 and 6 (TSFF and NTF's) can be designed to nitrify, limited operating data that support performance to the ammonia criterion of 2 mg/L have been provided by manufacturers of TSFF systems. However, NTF's have a longer operating record than TSFF systems, and that is why process reliability for NTF's systems is stated as "Good".

Visual impacts to the plant's neighbors to the north will be low, except for Alternative No. 4 BAF's, which have a high profile. Alternative No. 6 NTF's has a profile similar to the one-story building that will house the MF/RO system.

Conclusions

Based on the evaluation of the alternatives presented in this Supplement to TM-1, the following conclusions can be drawn:

1. Nitrifying Activated Sludge Alternative No. 1 does not provide reliable effluent quality of 2 mg/L ammonia for current average day flow rates.

2. Providing a reliable nitrifying activated sludge system by modifying the City's activated sludge system will be highly disruptive and result in a high capital and operating cost, compared with other available, stand-alone alternatives.

3. Three stand-alone tertiary, biological nitrification alternatives are capable of meeting the 2 mg/L ammonia criterion. Biological activated filters and nitrifying trickling filters have more proven performance as stand-alone nitrification systems, than do submerged fixed film systems.

4. BAF's have a high equipment profile of about 25 feet; they also have the highest capital and operating cost.

5. Alternative No. 6 Tertiary NTF's appears to be the most cost-effective alternative that can meet the ammonia criterion of 2 mg/L.

6. The estimated capital cost of Alternative No. 6 is within the Water Reuse Project budget allocation for nitrification for a 2 mgd project, as presented in the project cost estimate update, dated 8 March 2005.

7. Using a stand-alone nitrification system will avoid operational problems at the City's basic secondary treatment system during wet weather periods when it must accommodate high flows and still meets its NPDES permit requirements.

Recommendation

Based on the evaluations conducted and the conclusions reached in the performance of this study, CDM recommends that City staff, along with CDM, visit existing treatment plants that have stand-alone NTFs as their nitrification system (such as Sunnyvale, CA) to learn their operating characteristics and performance, and then determine if they are comfortable that this type of biological nitrification will consistently meet 2 mg/L ammonia.

1.0 Introduction and Purpose of the Technical Memorandum

1.1 Background

TM1, dated September 2004, evaluated several alternative methods of removing ammonia from the Benicia wastewater to make the treated effluent suitable for reuse at the Valero refinery. Options considered included removal by the water reuse treatment process (i.e., reverse osmosis) and biological process conversion at the City's WWTP. Four options were considered for the biological conversion of ammonia, as follows:

- Expansion of the activated sludge process
- Split flow between the rotating biological contactors (RBC's) and the activated sludge process
- RBC treatment of primary effluent prior to the activated sludge process
- A second stage nitrification system, using the RBC process

Based on the analyses performed in TM-1 regarding ammonia removal, it was concluded that it is more cost-effective to remove ammonia by biological nitrification than to utilize the reverse osmosis (RO) treatment process plus providing additional treatment to the RO concentrate and the blended permeate. Hence, CDM recommended that further analyses be performed to determine the most cost-effective, biological nitrification process.

1.2 Purpose of the Supplement

The purpose of this Supplement is to identify and screen potentially available biological nitrification technology options in order to determine the most feasible options. Conceptual designs for the most feasible options were prepared and analyzed for performance, reliability and economics in order to determine the most cost-effective alternative.

2.0 Development and Screening of Biological Nitrification Treatment Technologies

In order to provide additional assurance that the best nitrogen control technology is selected, an expanded list of biological treatment technologies that would potentially provide full-time nitrification was developed and screened. The technologies are listed in **Table 2-1**.

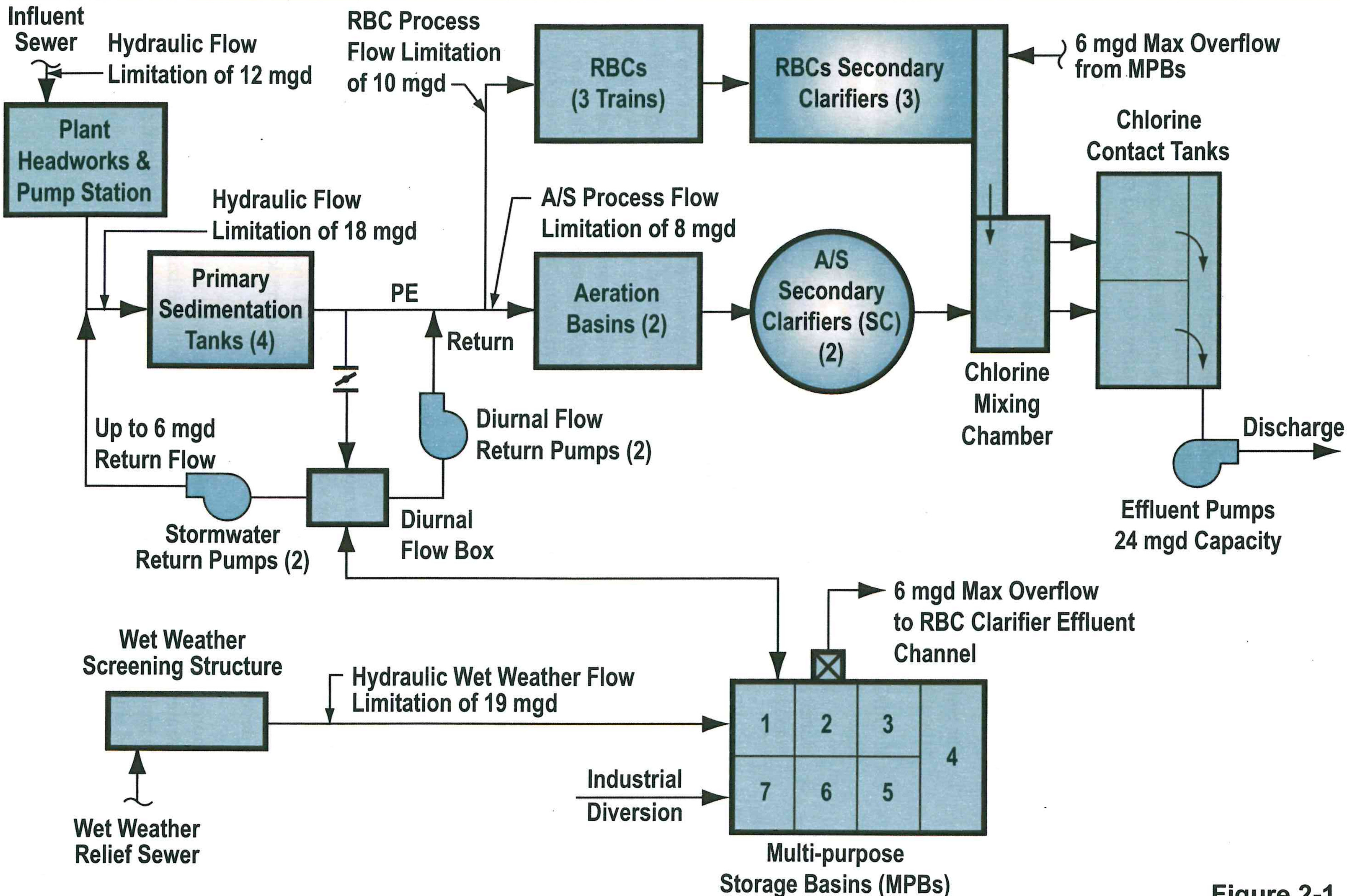


Figure 2-1
City of Benicia WWTP Existing Process Schematic Diagram

CDM

W:\REPORTS\Benicia\Benicia Supplement to TM1\Graphics\City of Benicia Existing Process.ai 12/02/05

In anticipation of wet weather flow conditions, the plant operations staff prepares one of the three RBCs to receive primary effluent flow in excess of 8 mgd. The preparation includes recirculating secondary effluent through one RBC train to grow the biological culture to treat the wet weather flow when necessary. Up to 10 mgd of primary effluent can be routed through the three RBC trains for processing wet weather flow.

Based on the above discussions, the follow requirements must be taken into consideration in developing nitrification alternatives that involve utilizing the existing activated sludge system in order for the plant to meet discharge requirements:

1. For wet weather operations, at least one RBC train must be available for processing wet weather flows.

2. If a RBC train is removed or otherwise not available for processing wet weather flows, then additional capacity must be provided in the activated sludge system.

3. Nitrification alternatives that would negatively impact the City's wet weather management program would be considered unacceptable.

Use of the RBCs for pre-treatment of dry weather flows would complicate or preclude their use for treatment of wet weather flows, would require intermediate pumping, and hence, will not be considered further. EHRC is an alternative to the current wet weather program, and is also eliminated as duplicating current plans. Nitrifier seeding is still an innovative technology and requires pilot testing or full-scale demonstration to verify its efficiency. IFAS systems require installation of relatively large volumes of plastic media or large frames for rope type media into the existing aeration basins. Since the use of IFAS media complicates maintenance of the aeration basins, and since there are only two aeration basins, IFAS will also not be considered further for modification to the existing activated sludge (A/S) system. However, a tertiary, stand-alone system using free-floating or rope type, fixed-film media is considered feasible.

Hence, from the list of treatment technologies options in **Table 2-1**, only options 1, 3, 4, 5 and 7 appear feasible at this time.

Options 1 and 3 involve extensive modifications to the City's existing WWTP. Options 4, 5 and 7 are basically stand alone systems, which can be sized solely to meet the flow demands of the Water Reuse Project. Whereas, Options 1 and 3 require that the entire secondary treatment system, along with wet weather operations, be included in the process development.

In addition to the five remaining options, we also present a sixth option, which demonstrates the reliability limitations associated with utilizing the existing two aeration basins and merely adding a third secondary clarifier and additional return activated sludge (RAS) pump.

3.0 Discussion of Process Considerations and Assumptions for Biological Nitrification Alternatives

Six biological nitrification alternatives were selected from the technologies discussed above for further analysis. These alternatives are listed in **Table 3-1**.

Table 3-1
Biological Nitrification Alternatives for Ammonia Removal

Alternative	***Description***
1	Expand existing activated sludge system – add a 3rd SC. Nitrifying Activated Sludge (2 AB's & 3 SC's)
2	Expand existing activated sludge system – add 3rd AB and a 3rd SC. Nitrifying Activated Sludge (3 AB's & 3 SC's)
3	Convert primaries to chemically enhanced primary treatment (CEPT) and add a 3rd secondary clarifier. Nitrifying Activated Sludge & CEPT
4	Add stand-alone tertiary nitrifying biological aerated filters. Nitrifying BAF's
5	Add stand-alone tertiary submerged, fixed-film nitrification system. TSFF Nitrification
6	Add stand-alone tertiary nitrifying trickling filters. Nitrifying TF's

3.1 Overview of Design Parameters for Nitrifying Activated Sludge Alternatives (Alternative Nos. 1, 2 & 3)

Alternative Nos. 1, 2 and 3 involve modifying the entire secondary treatment process to the nitrification mode. As a result, these alternatives must be designed to nitrify the entire plant flow under all expected conditions of influent flows and loads. A brief discussion of the nitrifying activated sludge process follows. A more extensive discussion along with additional design criteria assumptions are contained in **Appendix B**.

3.1.1 Description of Nitrifying Activated Sludge

Activated sludge aeration basins and final clarifiers must be evaluated together since the clarifiers must be able to adequately separate the mixed liquor suspended solids (i.e., biomass) grown in the aeration tanks. Secondary clarifier capacity depends on the hydraulic loading rate and the settling velocity of biomass in the mixed liquor suspended solids (MLSS). Critical design conditions are different for aeration tanks and clarifiers. Aeration basins are sized to provide the biomass inventory needed to treat the largest extended loads (both carbon and nitrogen for nitrifying activated sludge system) sent to the process, which are usually the loads of the average day during the maximum month. Because secondary clarifiers react very quickly to increased flows and solids loads, they are designed for the predicted maximum daily flows.

In order to achieve full nitrification in the activated sludge process, biomass retention times (aka SRT's), longer than those used for design of conventional activated sludge (AS) plants, are required. Also, sufficient process air supply must be provided to nitrify the entire portion of the influent total Kjeldahl nitrogen (TKN) that is converted to nitrate nitrogen by the nitrification process. Hence, additional secondary clarifiers, additional aeration basins, or both, and more blower capacity are usually required to upgrade existing plants from a conventional AS process, originally designed to remove only organic material (viz, BOD), to nitrifying activated sludge (NAS).

3.1.2 Update of Plant Nitrogen Loading

To develop the process requirements necessary to convert the City WWTP to NAS, a thorough understanding of the nitrogen loading on the AS process is required. After further examination of the plant data from January 2002 through May 2004, which were presented in TM-1, it was determined that the nitrogen loads in the plant's primary effluent should be re-evaluated because existing primary effluent data were not sufficient to evaluate thoroughly nitrification alternatives and sizing of new facilities.

Nitrogen enters a wastewater treatment plant in the raw wastewater as ammonia and organic nitrogen. Typically, the TKN load (which includes ammonia) is approximately twice the ammonia load. Organic nitrogen not removed in the primary clarifiers will be converted to ammonia in a conventional activated sludge process. At the City's WWTP, additional ammonia is generated by the anaerobic digestion process. Residual wastewater (filtrate) from the belt filter press (BFP) dewatering system is recycled back to the treatment system, carrying with it a significant amount of ammonia. Using computerized process simulation model (BioWin™) to generate a mass balance, CDM calculated that about 160 pounds per day (ppd) of ammonia is being returned to the process with the filtrate. This represents about 20% of the primary effluent TKN.

In February 2005, the City's plant staff, at CDM's request, collected and analyzed primary effluent (PE) TKN data for ten consecutive days to supplement the routine nitrogen measurements. It was found that the TKN concentration ranged between 23 and 37 mg/L. The lowest value occurred on Sunday, when the BFP is not typically running. The highest value occurred on Tuesday, which is when the highest BFP ammonia load impact is experienced.

The basic design criteria used in the development of the three NAS alternatives are presented in **Table 3-2.**

Table 3-2
Design Assumptions for Nitrifying Activated Sludge Alternatives Nos. 1, 2 & 3

Parameter	Units	Value for NAS Alt Nos 1 & 2	Value for Alt No. 3 - NAS & CEPT
Flow			
ADMM @ buildout	mgd	4.5	4.5
Peak, hour	mgd	8	8
Primary effluent			
BOD_5	mg/L	150	120
Total suspended solids, TSS	mg/L	125	75
Total Kjeldahl nitrogen, TKN	mg/L	40	36
Minimum temperature	^oC	17	17
Existing volume, 2 AB's	mgal	0.71	0.71
Existing surface area, 2 SC's	ft^2	7,700	7,700
Existing blower capacity (2 units operating; 1 standby)	icfm	3,000	3,000
Secondary effluent			
Maximum NH_4 concentration	mg/L	2	2

3.1.3 Overview of Design Parameters for NAS with Chemically Enhanced Primary Treatment (CEPT)

Chemically coagulating wastewater prior to clarification is the simplest enhancement that can be made to conventional primary clarification to increase overall secondary treatment capacity. The use of chemicals allows a higher peak overflow rate in the primary clarifiers during peak flow events while maintaining or increasing primary clarifier performance thus minimizing the clarifier surface area that must be provided for peak flows. **Figure 3-1** shows typical ranges of TSS removal for conventional primary sedimentation and chemically enhanced primary treatment versus overflow rate.

Most of the discussion above on nitrifying activated sludge applies also to Alternative No. 3, which includes CEPT. Design parameters used to size facilities for Alternative No. 3, NAS and CEPT are also presented in **Table 3-2**. As can be seen from a comparison between the values in **Table 3-2**, the BOD and TSS loadings of primary effluent are reduced as a result of the chemical additions.

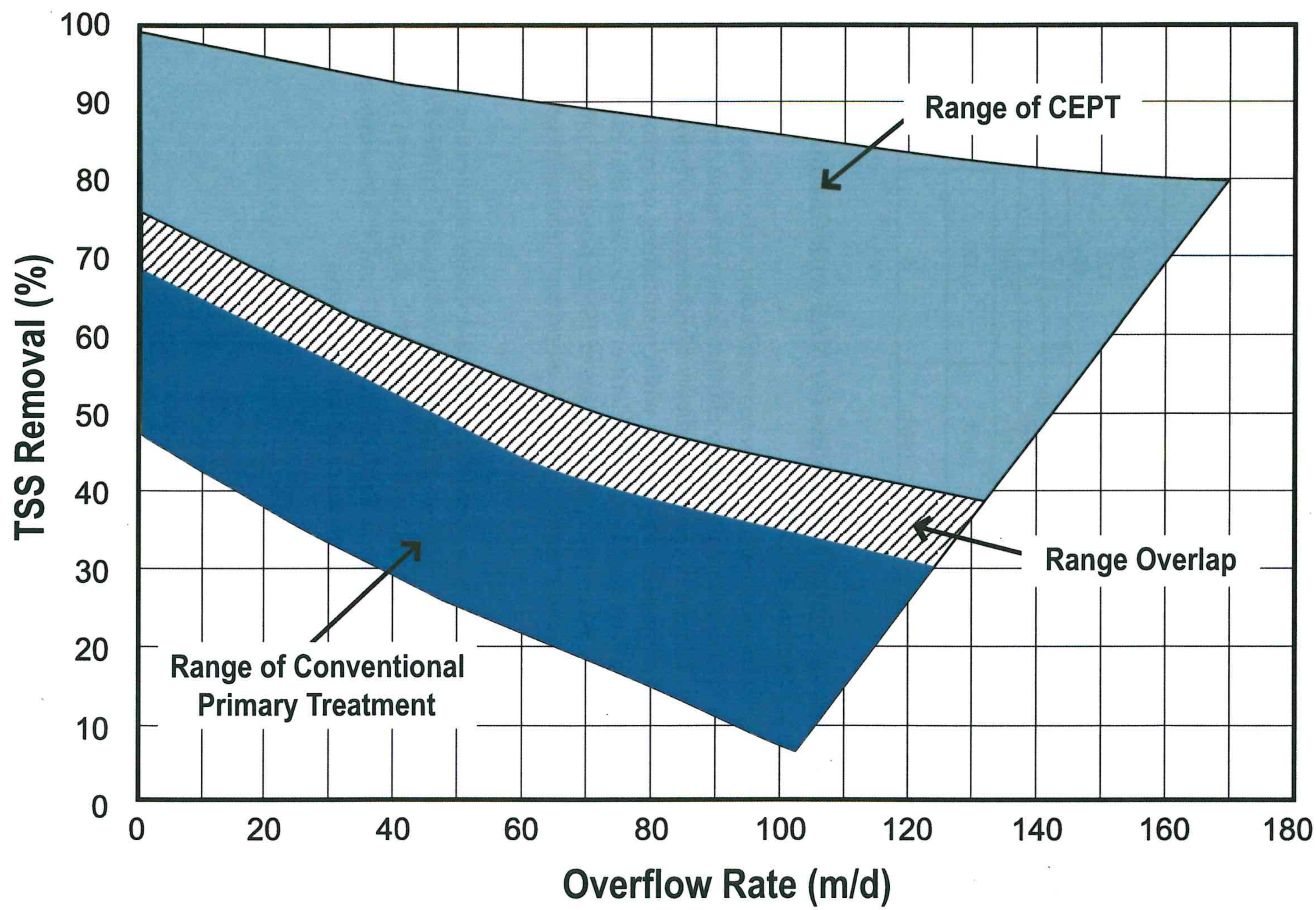


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Figure 3-1
Range of TSS Removal by Conventional and CEPT (from CDM/MW, 1995)

3.1.4 Discussion of Reliability Aspects of NAS Alternatives (Alternative Nos. 1, 2 & 3)

Process reliability can be defined as the ability of the treatment plant to perform as required under design conditions for a stated period of time. Most treatment plants are required by their NPDES permit to meet not to exceed values for key water quality parameters, on average, for set calendar periods (months and weeks, and sometimes as daily or instantaneous maximums). For Benicia the most stringent criteria for the biological treatment process is the monthly average limit. For this reason we have rated the capacity and reliability on the basis of the expected average day maximum month pollutant loads at the design flow.

Process reliability is closely linked to the reliability of the process equipment, and must be evaluated together with applicable equipment reliability standards. Treatment plants discharging to navigable waters that can be permanently or unacceptably damaged by discharge of effluent not meeting specified water quality criteria for only a few hours must typically be designed to meet EPA Class I equipment reliability criteria. EPA standards for equipment reliability are defined in the EPA publication *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability*, (EPA-430-99-74-001). According to the EPA reliability criteria a facility can be designed to treat less than design flow when one unit is out of service. Required capacity is typically reduced by a factor of 50 to 75 percent depending on the type unit. EPA Class I reliability criteria are summarized in **Table 3-3** along with our interpretation of design flow for each plant component.

Table 3-3
Summary of Pertinent EPA Class I Reliability Requirements

Plant Component	***EPA Class I Reliability Requirement***	***Interpretation Of Design Flow***
Hydraulic elements	Peak flow w/ largest unit out of service	Peak hour
Aeration system	Design transfer w/ largest unit out; backup may be uninstalled; minimum 2 installed units	Maximum day / maximum week
Biological treatment systems	Minimum 2 equal volumes; no backup required	Annual average / maximum month
Final clarifiers	75% design flow w/ largest unit out of service	Maximum day / peak hour

Process reliability for future nitrification for the Water Reuse Project at the Benicia plant is currently limited by the existence of only two aeration tanks and two clarifiers. Nitrification alternatives that add a third clarifier or a third aeration basin will improve the process reliability significantly. As discussed above clarifiers must be rated to handle maximum day and peak hour flow rates while aeration tanks are rated on their basis to treat maximum month pollutant loads.

For Alternative No. 1 to maintain consistent nitrification in the existing activated sludge process requires a MLSS concentration of about 3,000 mg/L at the existing average flow of 3.0 mgd. At this MLSS concentration, the final clarifier estimated capacity is about 12.6 mgd with all 3 units in service and no safety factor. With one clarifier down the capacity drops to about 8.4 mgd. Class I reliability requires a treatment capacity of 9 mgd (0.75 x 12 mgd). Loss of one aeration tank would require that the SRT be dropped to 3 days, and nitrification will likely stop. Thus Alternative No. 1 is not able to meet EPA equipment reliability criteria at existing, yet alone future, design flows as they pertain to the proposed ammonia limit of 2 mg/L, as input to the water reuse treatment system. However, because there is no ammonia limit for the discharge, the plant will be able to meet permit limits as long as flows in excess of 8 mgd can be bypassed around the activated sludge process. We understand that this is the design peak flow to the aeration basins under the new wet weather treatment system and operating scheme recently constructed.

For Alternative 2, the required MLSS concentration drops to about 2,100 mg/L at existing flows, and the final clarifier capacity becomes about 18 mgd with three clarifiers operating and 12 mgd with one unit down. At design conditions the required MLSS increases to about 3,200 mg/L, and the clarifier capacity drops to 12 mgd with three clarifiers in operation and about 8 mgd with one unit out. Again this is without a safety factor on clarifier performance. Alternative No. 2 provides significantly increased plant reliability in terms of both process and equipment reliability than Alternative No. 1. If an aeration basin is removed from service, the SRT reduces to 4 days, which is above the SRT wash out rate. If this occurs, nitrification performance will be reduced and the secondary effluent will likely not meet the 2 mg/L criterion, depending on the time of year and wastewater temperatures.

For Alternative No. 3, NAS with chemically enhanced primary treatment, it was assumed that the primary effluent BOD_5 can be reduced to about 100 mg/L. Under these process circumstances, an MLSS concentration of about 2,000 mg/L is required at existing flows and 3,000 mg/L at design flows. Under these conditions final clarifier capacity and reliability for Alternative 3 is similar to Alternative No. 2. Aeration tank reliability is still low since loss an aeration tank will require that the SRT to be reduced by half, and nitrification will likely stop depending on the time of year and wastewater temperatures.

In summary, of the three NAS alternatives, Alternative No. 2 provides the highest degree of reliability because nitrification can be maintained and required minimum wet weather flows can be passed with either one AB or one SC out of service. Alternative No. 3 provides less reliability than Alternative No. 2 because nitrification will likely be lost when one AB is removed from service; however, the activated sludge process can still pass the required wet weather flow with one SC out of service. Of the three full plant nitrifying activated sludge processes, Alternative No. 1 provides the least amount of reliability because loss of an AB will stop nitrification and loss of a SC will prevent the SCs from passing the required wet weather flow. **Table 3-4** presents a summary of the flow rates that each of these three alternatives can handle and still reliably meet the secondary effluent ammonia limit of 2 mg/L.

Table 3-4
Summary of Reliable Flow Limitation for Alternative No. 1, 2 & 3

Alternative	Average Day Max Month, mgd	Peak Hourly Flow, mgd
Estimated Flow at Build Out	4.5	8
Current Flow	3.7	8
Alt No. 1 – NAS with 2 AB's & 3 SC's	2.0 to 3.2	5.4 to 8.4
Alt No. 2 – NAS with 3 AB's & 3 SC's	3.2 to 4.0	8.3 to 10.8
Alt No. 3 – NAS & CEPT with 2 AB's & 3 SC's	2.4 to 3.6	6.4 to 9.9

3.2 Overview of Design Parameters for Biological Aerated Filters (BAFs)

BAFs are a type of attached growth biological treatment process that can be used for tertiary nitrification. BAFs have characteristics of both activated sludge systems and trickling filters. BAFs are similar to trickling filters in that the bacteria are grown attached to a media surface and the wastewater is passed over the media and the biofilm growing on it. Flow can be either upwards or downwards, although most BAFs for nitrification operate in an upflow mode. In a similar manner to activated sludge, air is provided by blowers and a diffuser system located near the bottom of a reactor full of wastewater. BAFs can be viewed as flooded trickling filters with a diffused aeration system.

BAFs can be an attractive nitrification technology for plants with limited land area or only have a need to nitrify a portion of the plant flow. Both conditions apply at the Benicia wastewater plant. Oxygen transfer efficiency in BAFs is very high so the use of BAFs significantly reduces the incremental power needed for nitrification over the nitrifying activated sludge process. However, the requirement for pumping offsets a portion this advantage.

The configuration of biological filters is similar to conventional gravity filters with the main differences being the provision for aeration, and the size and depth of the filter media. Biological filters do provide some removal of suspended solids by filtration; however, they are primarily biological reactors. Media for biological filters is typically larger (1-4 mm) than the sand and anthracite used in conventional wastewater filters. Since the media provides a high specific surface area (230 m^2/m^3), the size of the reactor required is significantly reduced. Biological filter media is less dense than the sand media used in wastewater filters with specific gravities of 1.5 or lower as compared with the 2.65 specific gravity of filter sand and the 1.35-1.75 specific gravity of anthracite. BAFs require a relatively large amount of mechanical equipment in the form of pumps, blowers, diffusers and related controls. Even though BAFs are mechanically more complex than activated sludge, operation can be simple particularly when the cleaning cycles are automated through the use of programmable logic controllers. Due to the relatively high head loss across BAFs, influent pumping is required. Periodic backwashing is required to remove accumulated solids. Spent backwash, equivalent to approximately five percent of the applied flow, is returned to the head of the plant for reprocessing.

There are two main manufacturers of BAF's in North America: Kruger Incorporated (Veolia Water Systems), whose system is called Biostyr®; and Infilco Degremont Incorporated (IDI), whose system is called BIOFOR®. A list of IDI installations is contained in **Appendix B**.

The basic process design assumptions for Alternative No. 4 Nitrifying BAF's are presented in **Table 3-5**. **Figure 3-2** contains a process schematic of the BAF process. **Figure 3-3** contains a diagram of the BIOFOR® BAF process.

Table 3-5
Design Assumptions for Alternative No. 4 Nitrifying BAF's

Parameter	Units	Value for BAF's
Design Flow (constant)	mgd	2.6
Secondary Effluent		
BOD_5	mg/L	25
Total Suspended Solids, TSS	mg/L	30
Ammonia Nitrogen,	mg/L	30
Minimum temperature	^oC	17
Ammonia Mass Loading	$kgNH_4\text{-}N/day/m^3$	1.0
Average Hydraulic Loading	gpm/sf	4
Tertiary Effluent		
Maximum NH_4 concentration	mg/L	2

3.3 Overview of Design Parameters for Tertiary Submerged Fixed-Film Reactor Systems

Tertiary submerged fixed-film reactor systems are composed of a reaction vessel and either fixed or moving-bed media on which nitrifying bacteria are grown. Air is diffused into the water-media culture. Fixed media consist of either ropes that are attached to frames, or plastic crates, similar to those used in packed bio-towers. Moving-bed media are made of either sponges or small plastic elements. Since maintenance of the fixed media have presented challenges, only plastic media of the moving-bed type will be considered in this evaluation. Two manufacturers furnish their particular patented plastic media. The bacteria culture (biofilm) grows on plastic media. The core of the process is the biofilm carrier elements that are made from polyethylene with a density slightly below that of water. The elements are designed to provide a large protected surface area for the biofilm and optimal conditions for the bacteria culture when the elements are suspended in water. For nitrification, the reactor vessel is filled from approximately 30 to 50 percent of volume with the media, or carrier elements.

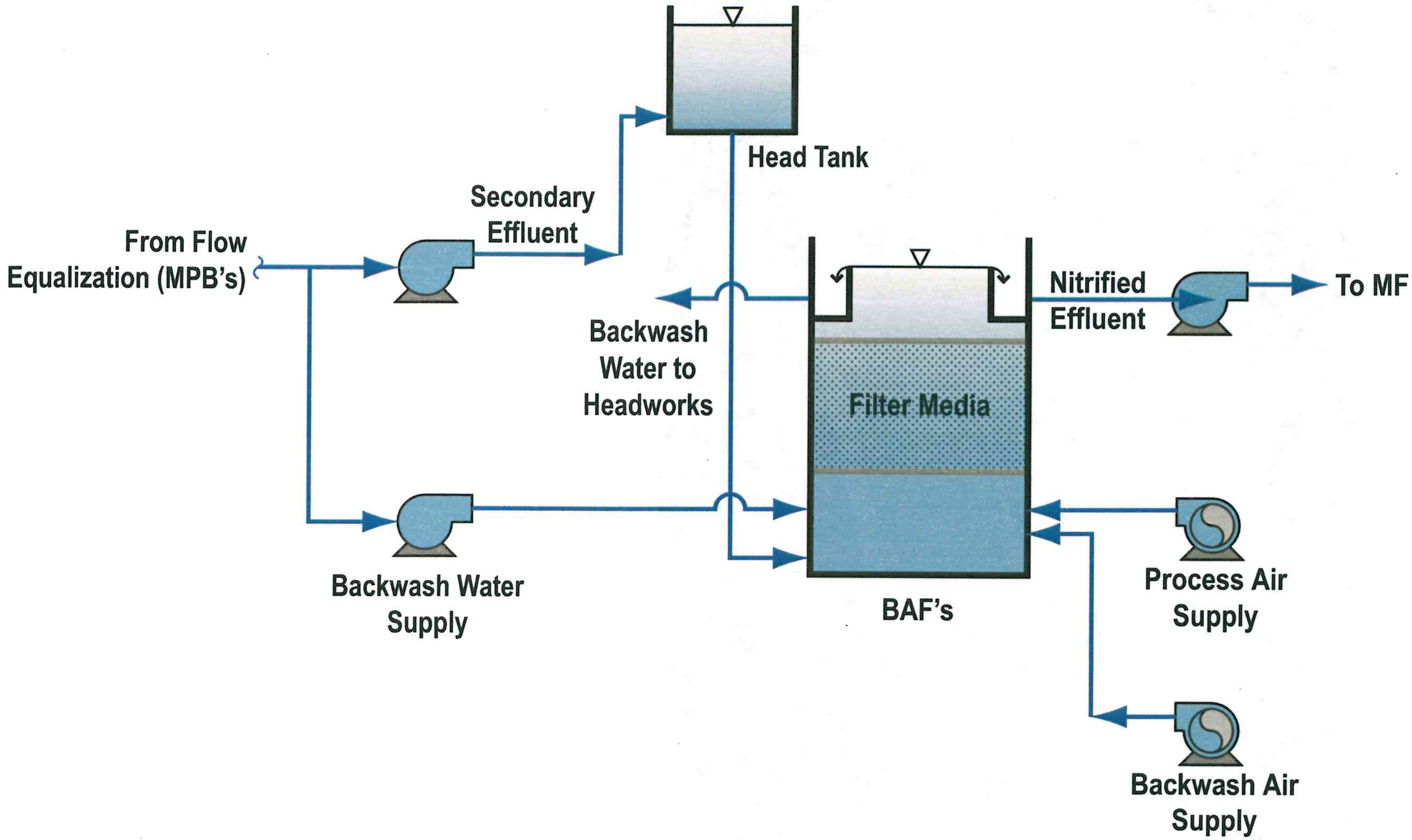

Figure 3-2
Schematic of BAF Nitrification System

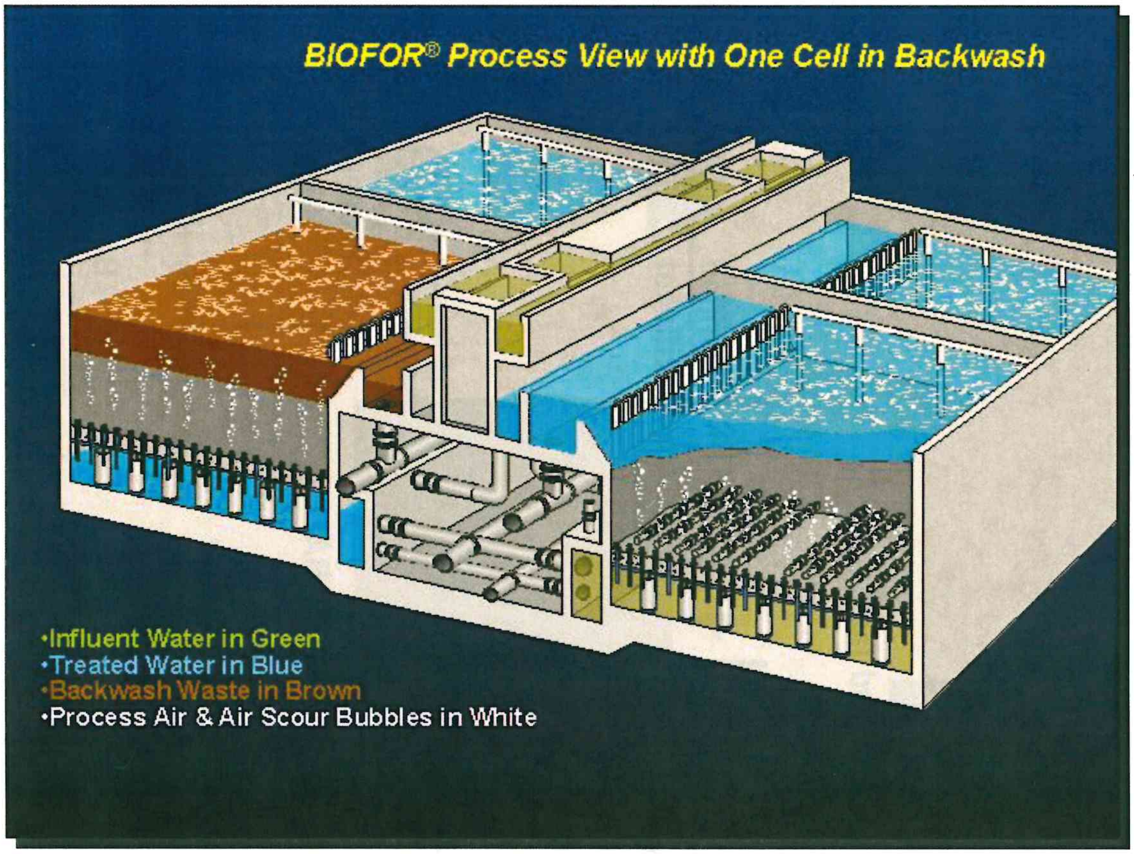

Figure 3-3
Process Diagram of IDI's BIOFOR® BAF Process

The two manufacturers considered for Benicia (AnoxKaldnes from Norway and Hydroxyl, now owned by IDI) have each developed its own carrier with different shape and size.

Some of the benefits of using this type of system include the following:

- Compact and thus small footprint
- Stable also under large load variations
- Flexibility, in that almost any shape of reactor can be utilized; provides for use of existing tanks for bioreactors

Other features of moving bed bioreactors for nitrification include: no sludge return, no clogging of reactors and, depending on requirements of downstream processes, no particle separation stage, such as clarifiers or filters.

The basic process design assumptions for Alternative No. 5 TSFF Nitrification are presented in **Table 3-6**. A partial list of TSFF installations is contained in **Appendix C**. **Figure 3-4** shows a schematic of the TSFF Nitrification process.

Table 3-6

Design Assumptions for Alternative No. 5 TSFF Nitrification

Parameter	Units	Value for TSFF
Design flow (constant)	mgd	2.55
Secondary effluent		
BOD_5	mg/L	25
Total suspended solids, TSS	mg/L	30
Ammonia nitrogen,	mg/L	30
Minimum temperature	^oC	17
Ammonia mass loading	$kgNH_4$-N/day/m^3	0.4
Tertiary effluent		
Maximum NH_4 concentration	mg/L	2

3.4 Overview of Design Parameters for Tertiary Nitrifying Trickling Filters

Nitrifying trickling filters are attached growth biological treatment processes that allow the bacteria providing treatment to grow on the surface of solid media, as the wastewater being treated flows over the media. This is opposite of the suspended growth processes (i.e., NAS, as in Alternative Nos. 1, 2 & 3 and TSFF systems, as in Alternative No. 5) where the bacteria are "suspended" in the wastewater. Like the suspended growth processes, trickling filters can be used to nitrify as a separate, tertiary process or in combination with carbonaceous BOD removal. Trickling filters used for tertiary wastewater treatment for ammonia removal are called nitrifying

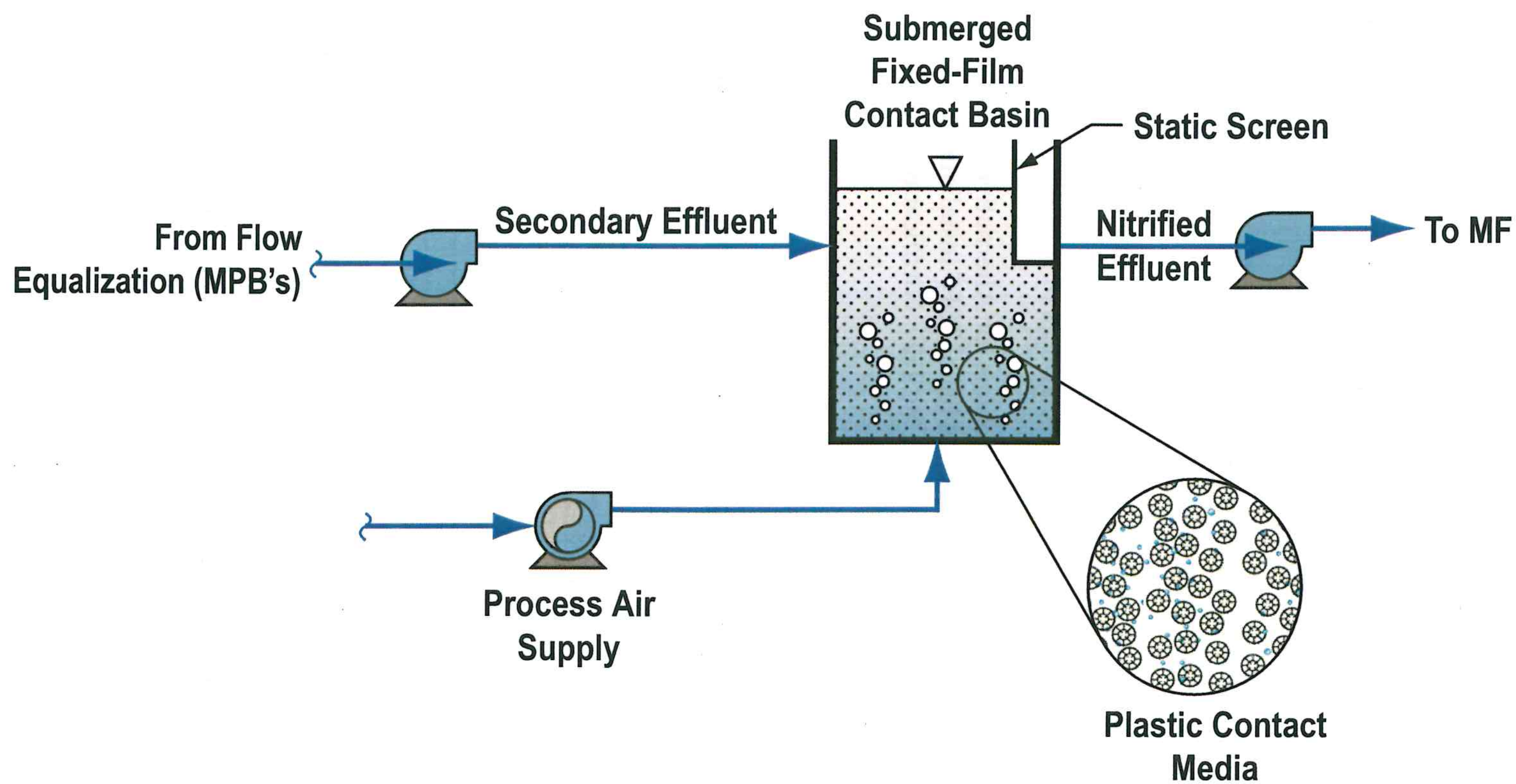

Figure 3-4
Schematic of Teritiary Submerged, Filxed-Film Nitrification System

trickling filters or NTF's for short. In order for the process to nitrify successfully, the BOD5 concentration needs to be below about 20 mg/l. In nitrifying trickling filters (NTF's) the wastewater is distributed over a tower filled with trickling filter media, allowed to flow down the depth of the media, and then recollected and directly discharged. Trickling filter media can be rock, wood or grids of plastic sheets constructed into self-supporting blocks.

Advantages of NTFs include their simplicity of operation and low energy requirements. Few operational adjustments are possible; recycle pumps, rotary distributors and induced air fans are the only mechanical equipment. Oxygen transfer occurs as the wastewater trickles down the media. Enclosure of the media with a structure is required only for aesthetic reasons or if the ability to flood the media is desired. Low pressure, high volume fans or blowers are sometimes used to provide adequate oxygen during all climatic conditions. Nitrifiers produce relatively small amounts of solids, so NTFs do not require final clarifiers but the effluent solids concentrations are higher than a sand filter. Nuisance and predatory organisms including flies and snails can sometimes grow in NTFs, so designs provisions are made to avoid or control these pests. NTFs using plastic media are generally tall (20 ft), but they can also be designed at shallower depths.
The manufacturer that supplies the bulk of the plastic, high specific surface media for trickling filters is Brentwood Industries. The media comes in "baskets" about 4 ft by 4 ft by 8 ft long.

The basic process design assumptions for Alternative No. 6 Tertiary Nitrifying Trickling Filters are presented in **Table 3-7**. A partial list of installations of NTF's is contained in **Appendix D**. **Figure 3-5** shows a schematic of the TSFF Nitrification process.

Table 3-7
Design Assumptions for Alternative No. 6 Tertiary Nitrifying Trickling Filters

Parameter	*Units*	*Value for TNTF's*
Design flow (constant)	mgd	2.55
Secondary effluent		
BOD_5	mg/L	25
Total suspended solids, TSS	mg/L	30
Ammonia nitrogen,	mg/L	30
Minimum temperature	^{o}C	17
Ammonia mass loading	gm NH_4-N/day/m^2	1.3
Plastic media specific surface area	m^2/m^3	150
Tertiary effluent		
Maximum NH_4 concentration	mg/L	2
Recycle ratio	%	50

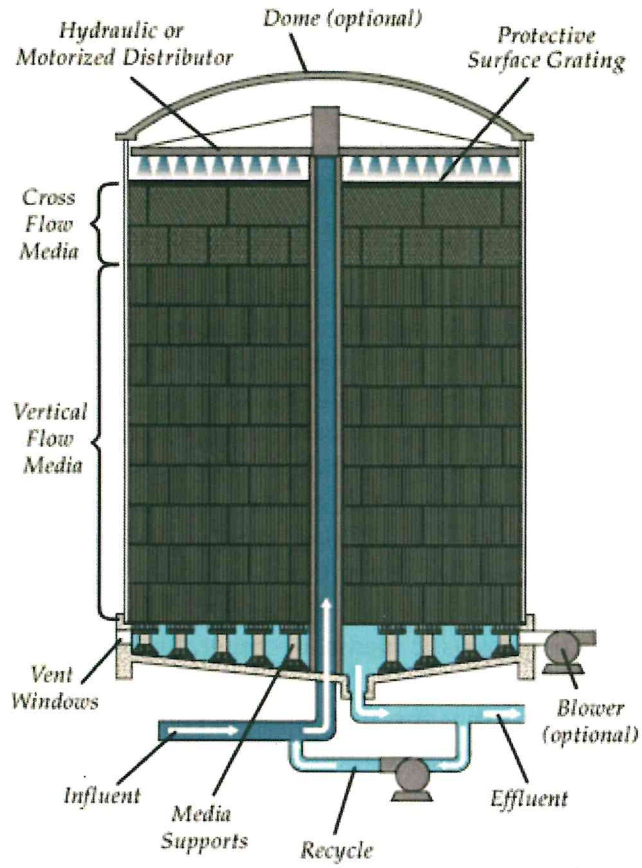

Figure 3-5
Process Schematic of Tertiary Nitrifying Trickling Filters

4.0 Conceptual Design of Biological Nitrification Alternatives

Using the basic design assumptions and criteria, presented in the above tables and the additional information contained in **Appendix A**, conceptual designs of the six biological nitrification alternatives were developed and are presented in the information that follows.

4.1 Alternative No. 1 Nitrifying Activated Sludge (2 AB's & 3 SC's)

Implementation of Alternative No. 1 requires the facility modifications and additions listed in **Table 4-1**. **Figure 4-1** presents a schematic diagram of this alternative.

Table 4-1
Facilities Required for Alternative No. 1 Nitrifying Activated Sludge (2 AB's and 3 SC's)

Item	*Description*
Add 3rd secondary clarifier	70' dia, 14' swd, hydraulic suction type sludge collector
Add 3rd RAS pump	15 hp, 1.5 mgd
Yard piping	
Replace all 3 blowers	3,000 icfm @ 9.1 psig, each
Add caustic feed system	Required for alkalinity control

4.2 Alternative No. 2 Nitrifying Activated Sludge (3 AB's & 3 SC's)

Implementation of Alternative No. 2 requires the facility modifications and additions listed in **Table 4-2. Figure 4-2** presents a schematic diagram of this alternative.

Table 4-2
Facilities Required for Alternative No. 2 Nitrifying Activated Sludge (3 AB's & 3 SC's)

Item	*Description*
Add 3rd secondary clarifier	70' dia, 14' swd, hydraulic suction type sludge collector
Add 3rd RAS pump	15 hp, 1.5 mgd
Add 3rd aeration basin	20' x 66' x 18' swd
Air diffusers & piping	
Add caustic feed system	Required for alkalinity control
Yard piping	
Replace blowers (3)	3,000 icfm @ 9.1 psig, each
Demolish one train of RBC's	Western most train

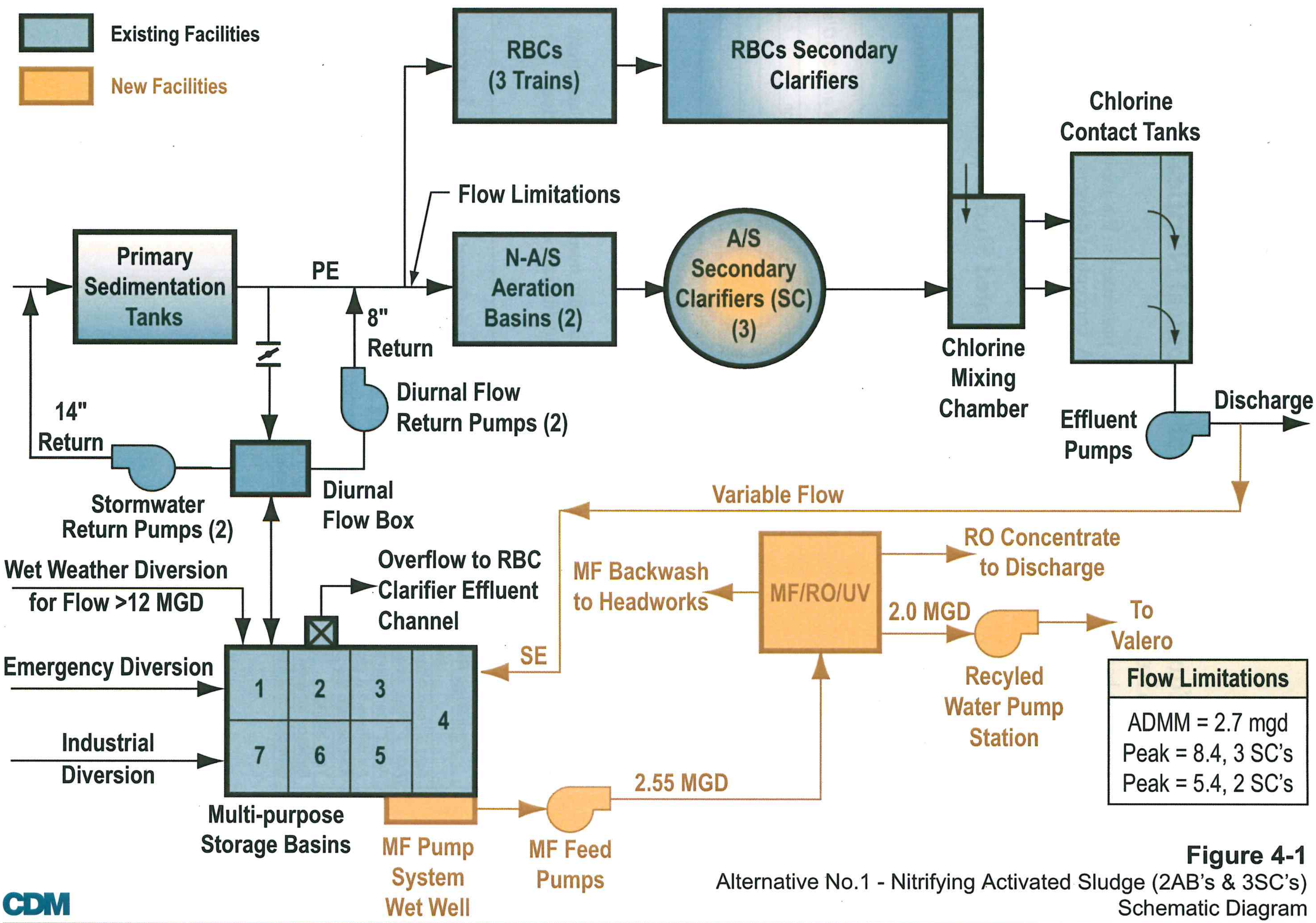

Figure 4-1
Alternative No.1 - Nitrifying Activated Sludge (2AB's & 3SC's)
Schematic Diagram

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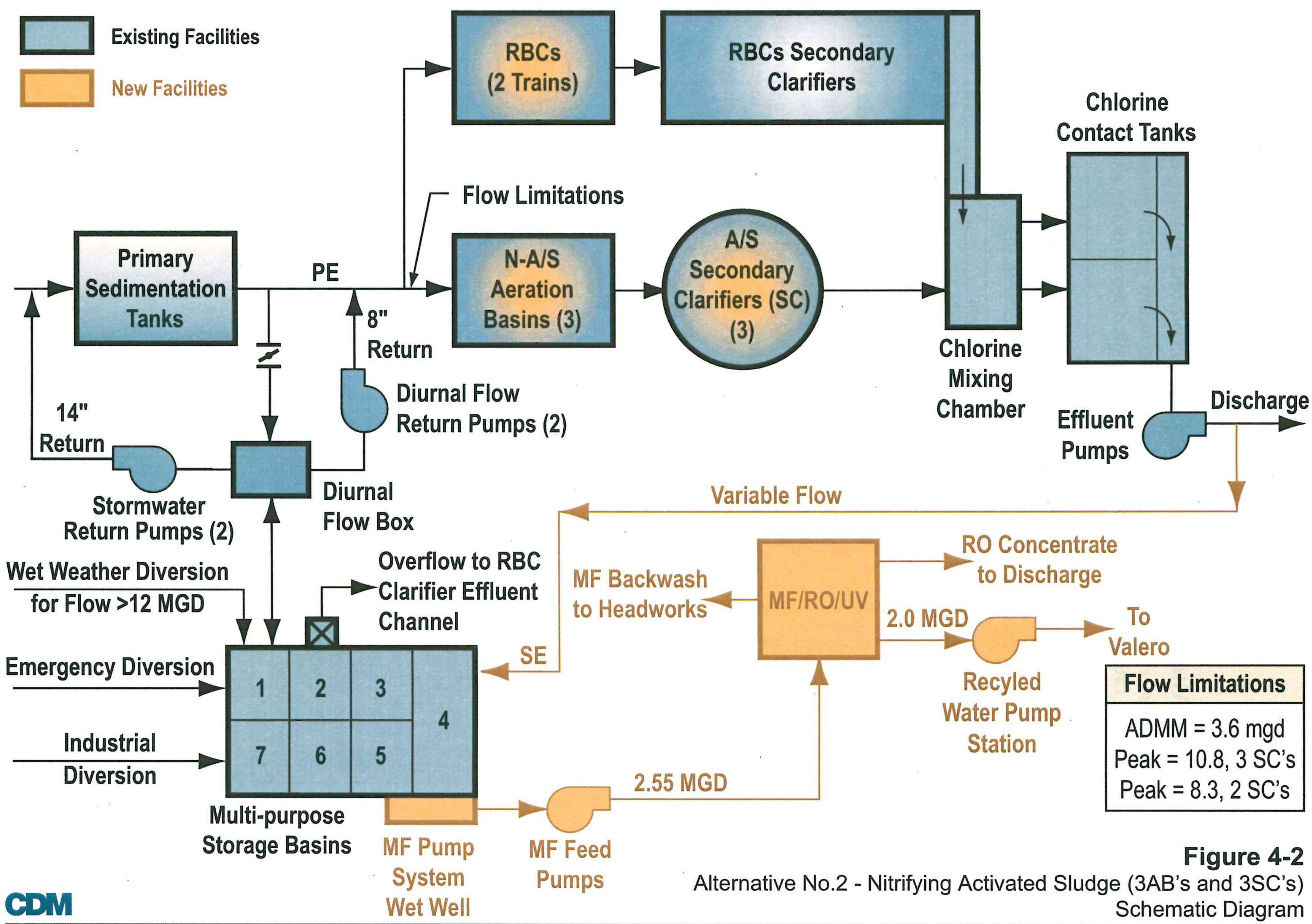

Figure 4-2
Alternative No.2 - Nitrifying Activated Sludge (3AB's and 3SC's)
Schematic Diagram

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4.3 Alternative No. 3 Nitrifying Activated Sludge & CEPT

Implementation of Alternative No. 3 requires the facility modifications and additions listed in **Table 4-3.** **Figure 4-3** presents a schematic diagram of this alternative.

Table 4-3
Facilities Required for Alternative No. 3 – Nitrifying Activated Sludge & CEPT

Item	*Description*
Add 3rd Final clarifier	70' dia, 14' swd, hydraulic suction type sludge collector
Add 3rd RAS pump	15 hp, 1.5 mgd
Replace all 3 blowers	3,000 icfm @ 9.1 psig, each
Add caustic feed system	Required for alkalinity control
Chemical Storage and Feeding system	Ferric Chloride feed pumps (1 duty/1 standby) and 2,000 gal storage tank

4.3.1 Locations of NAS and Water Reuse Project Facilities at Benicia WWTP

Figure 4-3A contains a conceptual site plan of the Benicia WWTP with the nitrifying activated sludge facilities for Alternative Nos. 1, 2 & 3 along with the proposed locations for the advanced treatment facilities of MF, RO and UV. The City's plant has been designed to accommodate the third secondary clarifier. As previously mentioned, one train of RBC's must be demolished in order to construct the third aeration basin.

4.4 Alternative No. 4 Nitrifying BAF's

Implementation of Alternative No. 4 requires the facility modifications and additions listed in **Table 4-4.** **Figure 4-4** presents a schematic diagram of this alternative. A conceptual site plan of Alternative No. 4 is shown in **Figure 4-4A.**

Table 4-4
Facilities Required for Alternative No. 4 – Nitrifying BAF's

Item	*Description*
BAF feed pumps	2 at 1,930 gpm (2.75 mgd) @ 45 ft TDH (1 duty, 1 standby)
Influent Screens	2 at 2,000 gpm each (1 duty, 1 standby)
BAF (2 cells)	427 sf each x 24 ft high
Process air blowers	2 at 250 scfm @ 11.5 psig, each (2 duty, 1 standby)
Air diffusers & piping	
Backwash supply pumps	3,500 gpm
Backwash blowers	2 at 1,130 scfm @ 10.5 psig, each (1 duty, 1 standby)
Air system cleaning pump	1 at 1,300 gpm @ 120 ft TDH
Add caustic feed system	Required for alkalinity control
Blower Building	Approximately 900 sf
Yard Piping	

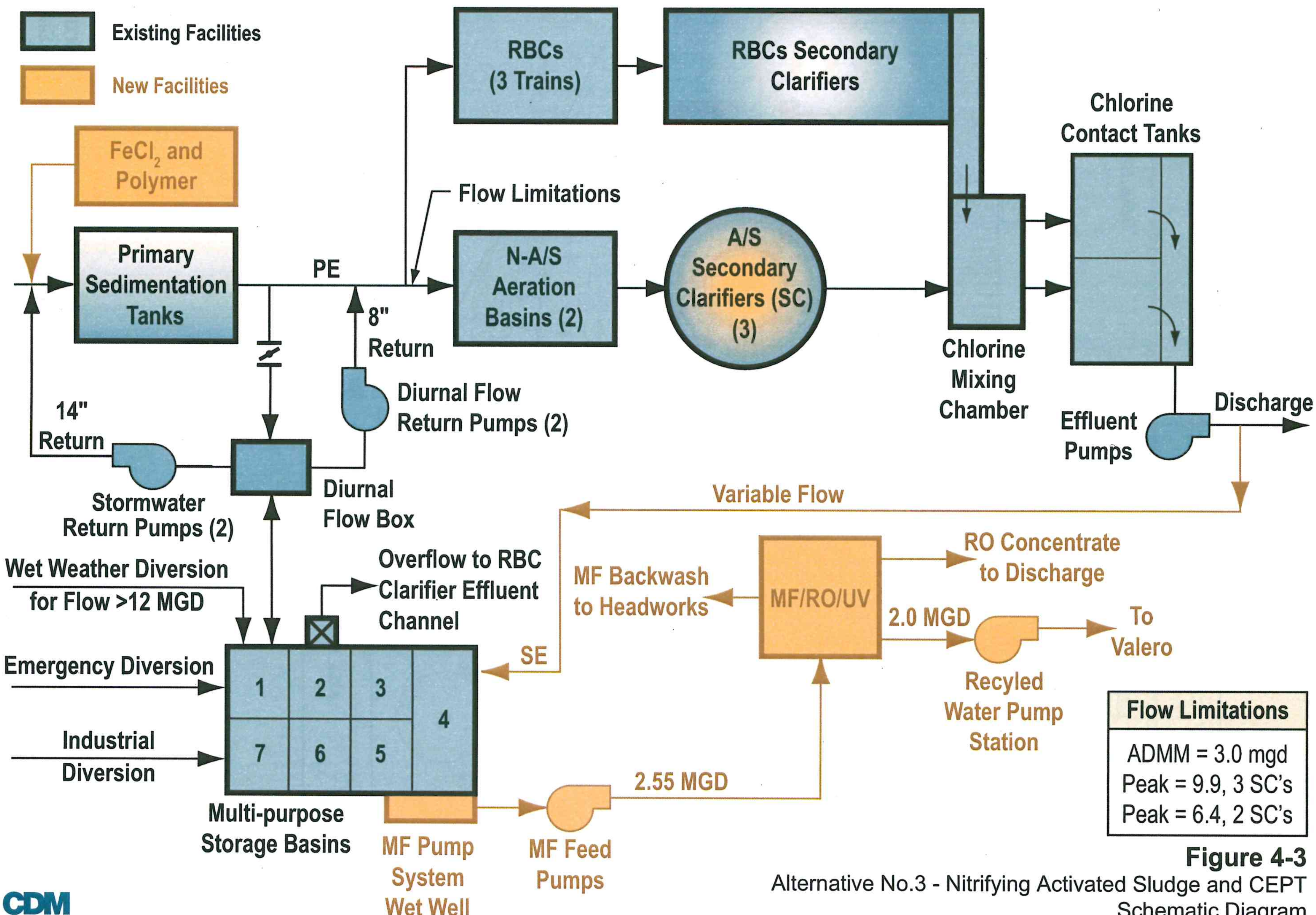

Figure 4-3
Alternative No.3 - Nitrifying Activated Sludge and CEPT Schematic Diagram

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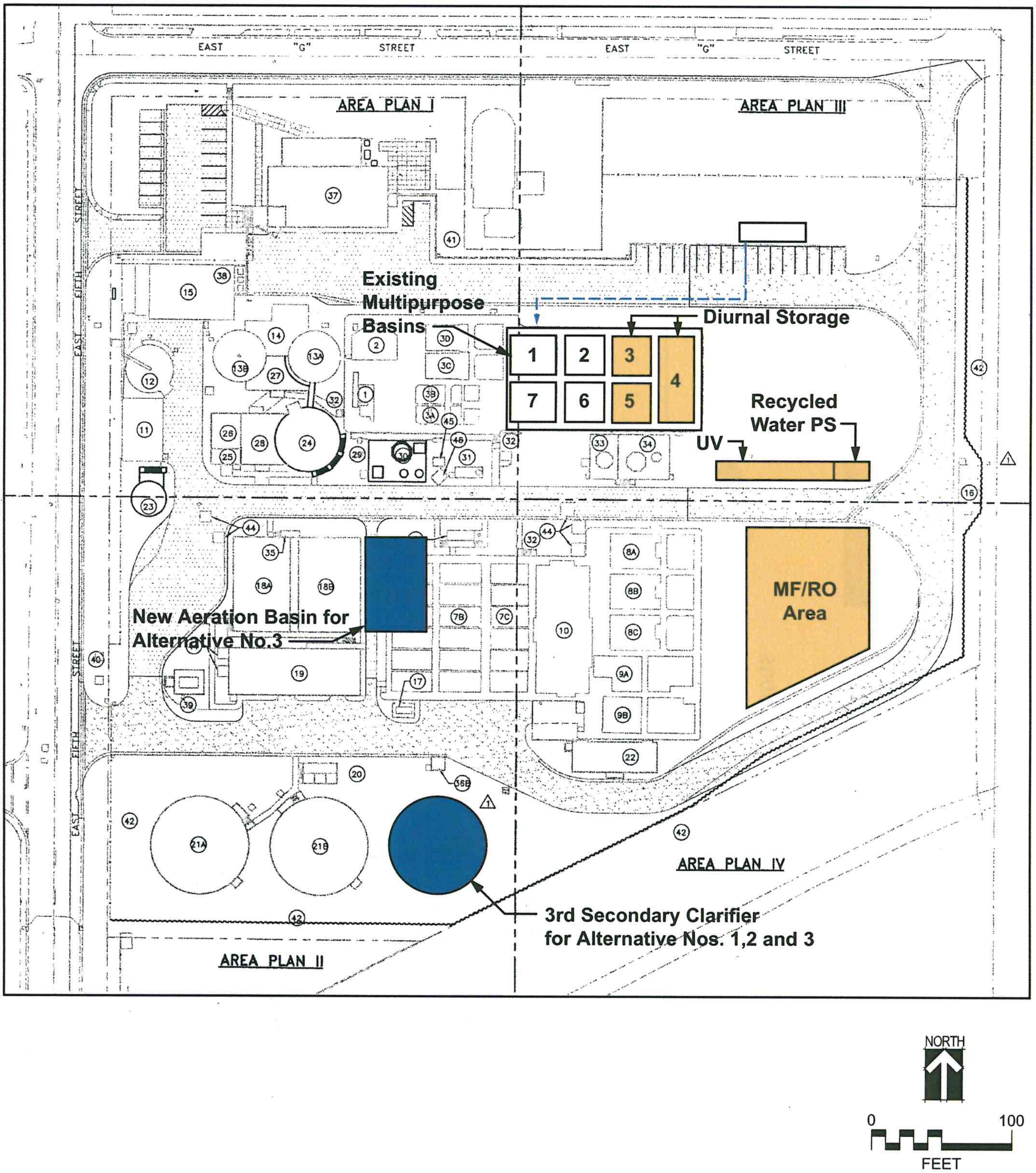

Figure 4-3A
Alternative Nos. 1, 2 and 3 Conceptual Site Plan
for NAS Alternatives

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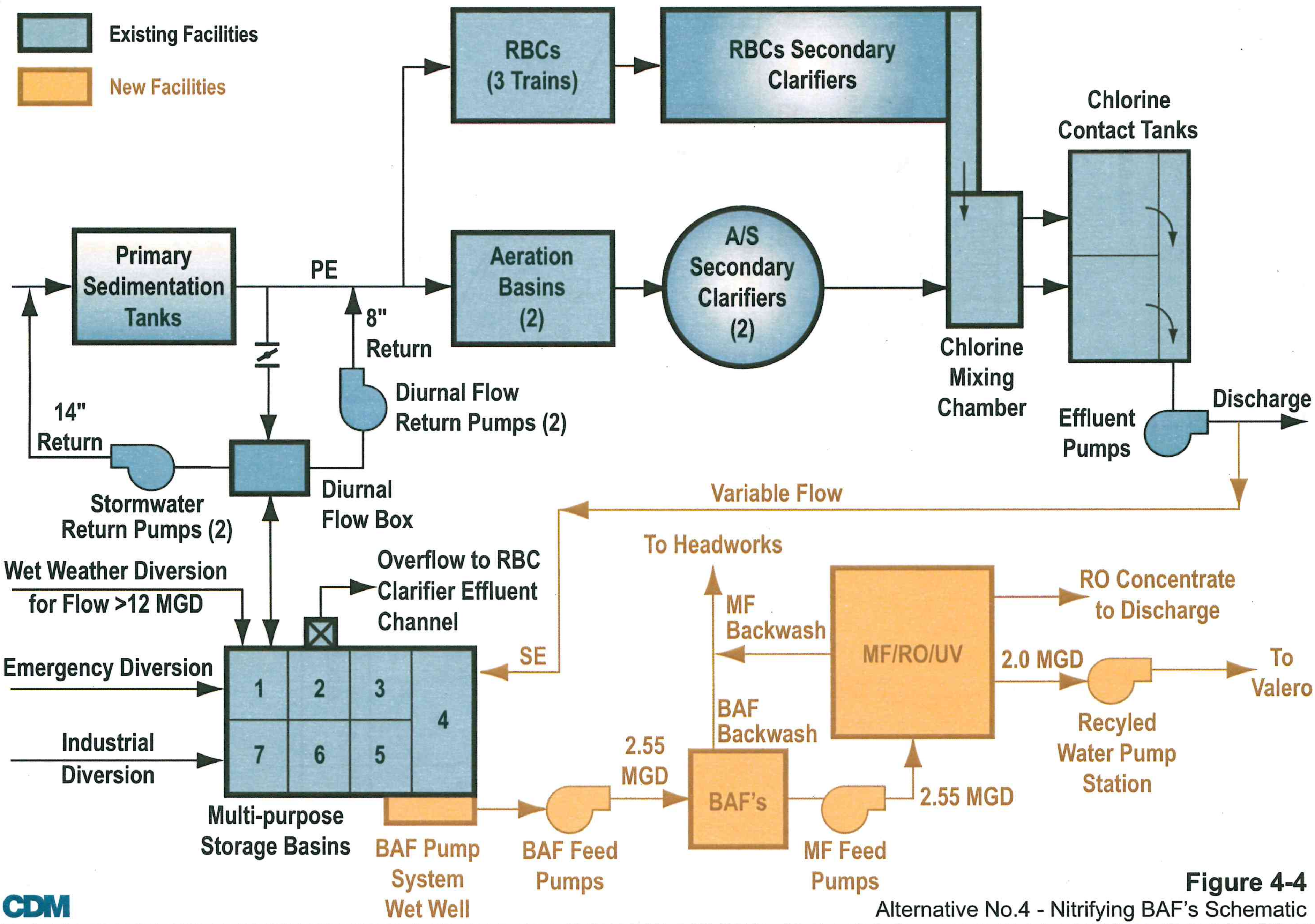

Figure 4-4
Alternative No.4 - Nitrifying BAF's Schematic

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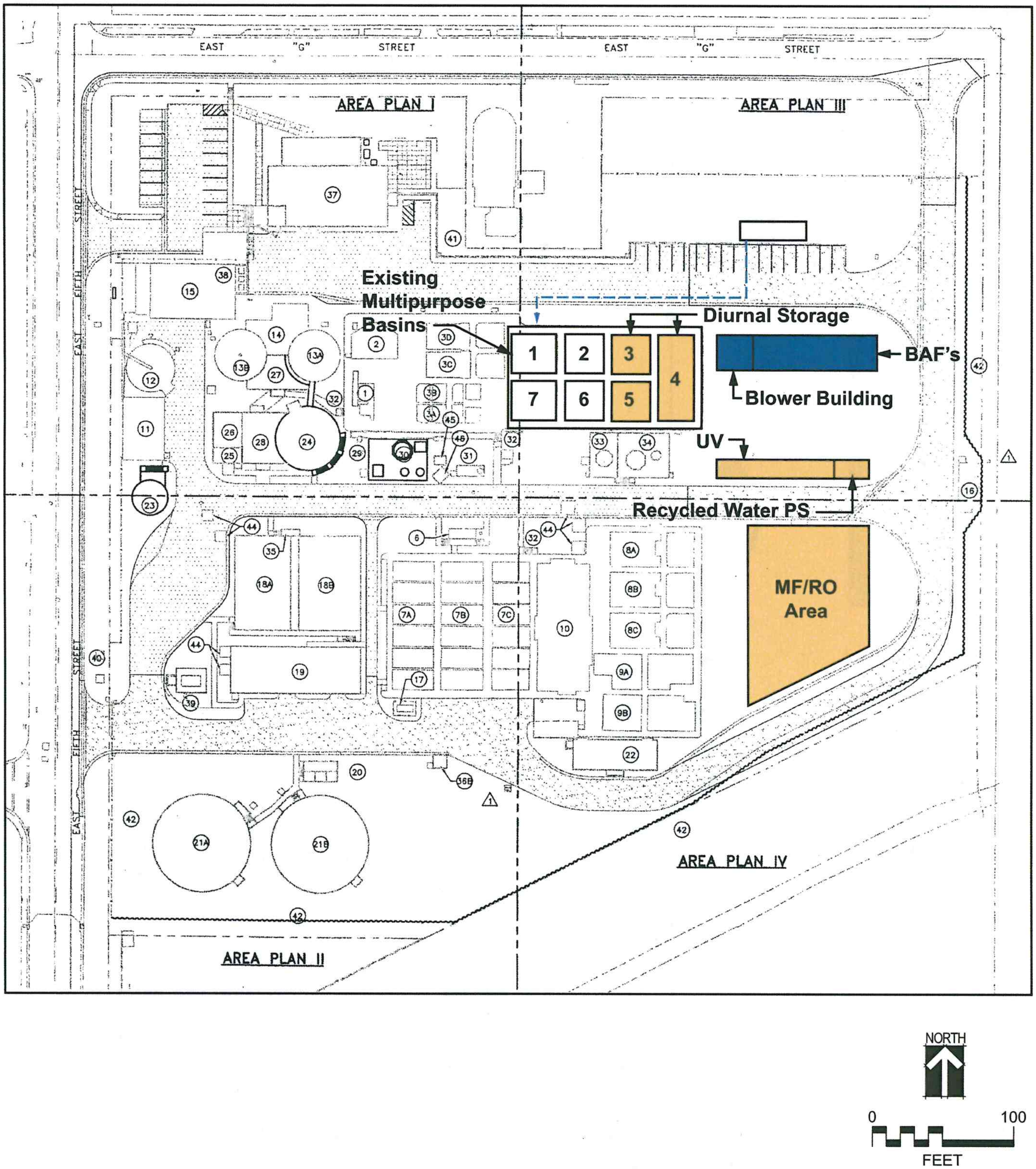

Figure 4-4A
Alternative No.4 Conceptual Site Plan for Nitrifying BAF's

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4.5 Alternative No. 5 TSFF Nitrification

Implementation of Alternative No. 5 Tertiary Submerged, Fixed-Film Nitrification requires the facility modifications and additions listed in **Table 4-5. Figure 4-5** presents a schematic diagram of this alternative. As conceptual site plan of the alternative is contained in **Figure 4-5A.**

Table 4-5
Facilities Required for Alternative No. 5 TSFF Nitrification

Item	***Description***
Feed pumps (2)	2.5 mgd, each
Influent Screens (2)	2 at 2,000 gpm each (1 duty, 1 standby)
Reactor Basins- 2 cells in series	50 ft x 50 ft x 15 ft swd, each
Media	37,500 cf (50% of reactor volume)
Static Effluent Screens	
Air Diffusers & piping	
Process air blowers (2)	3,000 scfm, at 8.3 psig, each
Add caustic feed system	Required for alkalinity control
Blower Bldg	600 sf
Yard piping	

4.6 Alternative No. 6 Tertiary Nitrifying Trickling Filters

Implementation of Alternative No. 6 Tertiary Nitrifying Trickling Filters requires the facility modifications and additions listed in **Table 4-6. Figure 4-6** presents a schematic diagram of this alternative. As conceptual site plan of the alternative is contained in **Figure 4-6A.**

Table 4-6
Facilities Required for Alternative No. 6 Tertiary Nitrifying Trickling Filters

Item	***Description***
Secondary effluent transfer pumps (2)	2.5 mgd, each
Trickling filters (2)	42 ft diameter x 12 ft media depth
Media	34,000 cf cross flow media
Static effluent screen w/ auger	
Process air blowers (8)	1,500 scfm, at 2-in H2O column (4 per filter)
Add caustic feed system	Required for alkalinity control
Combined feed and recycle pumps (2)	1,350 gpm @ 20 ft TDH
Yard piping	

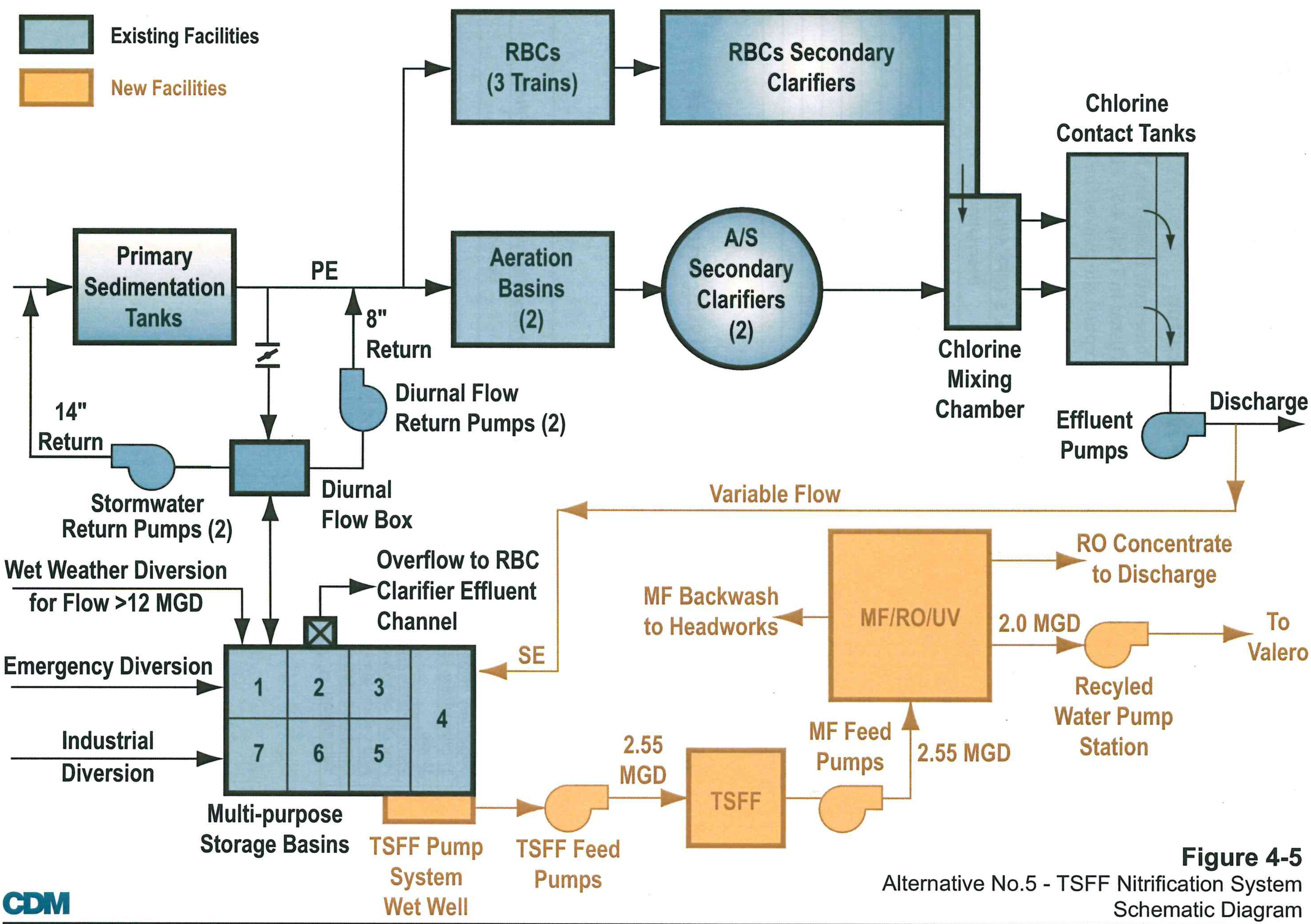

Figure 4-5
Alternative No.5 - TSFF Nitrification System
Schematic Diagram

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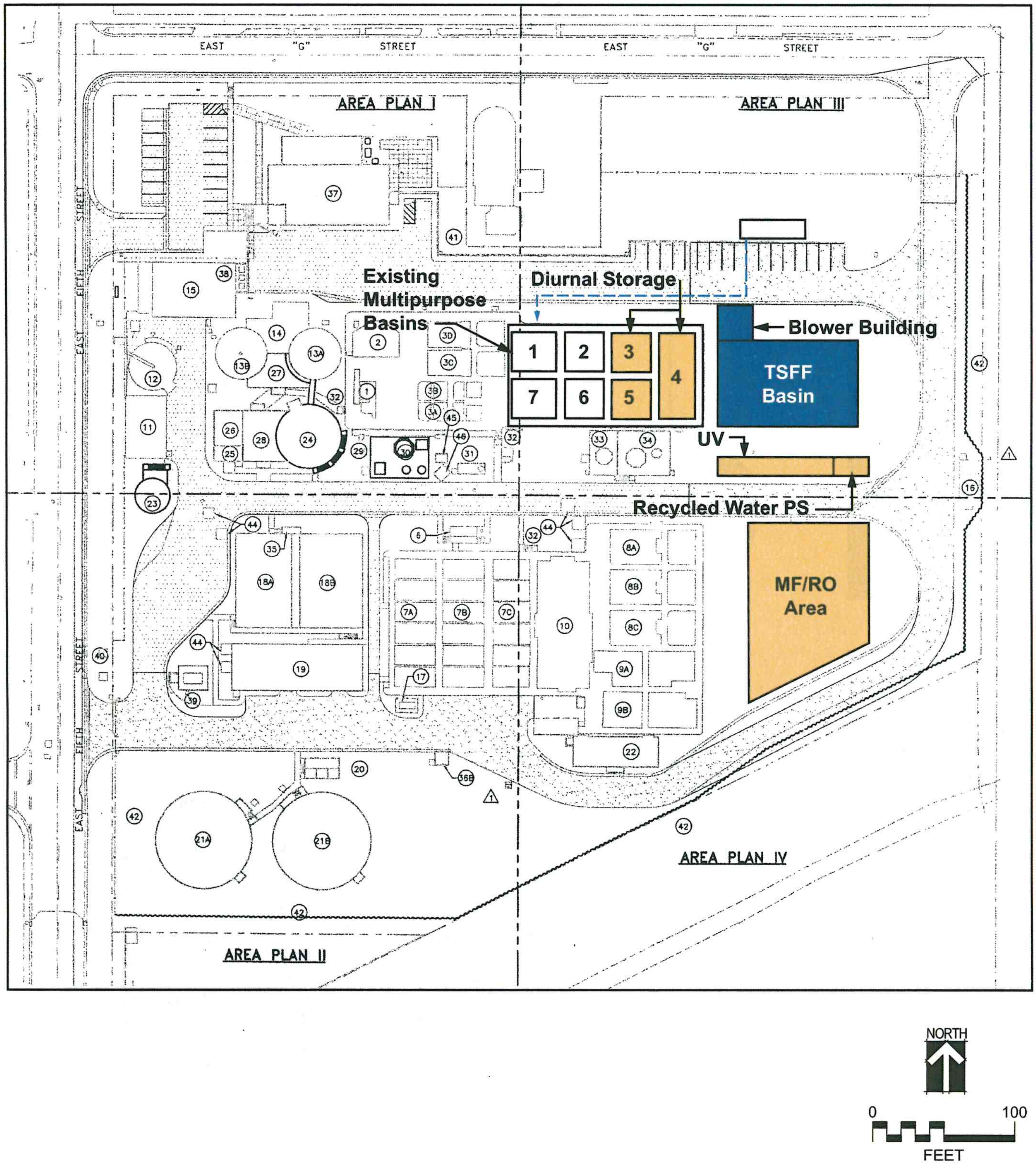

Figure 4-5A
Alternative No.5 Conceptual Site Plan
for TSFF Nitrification

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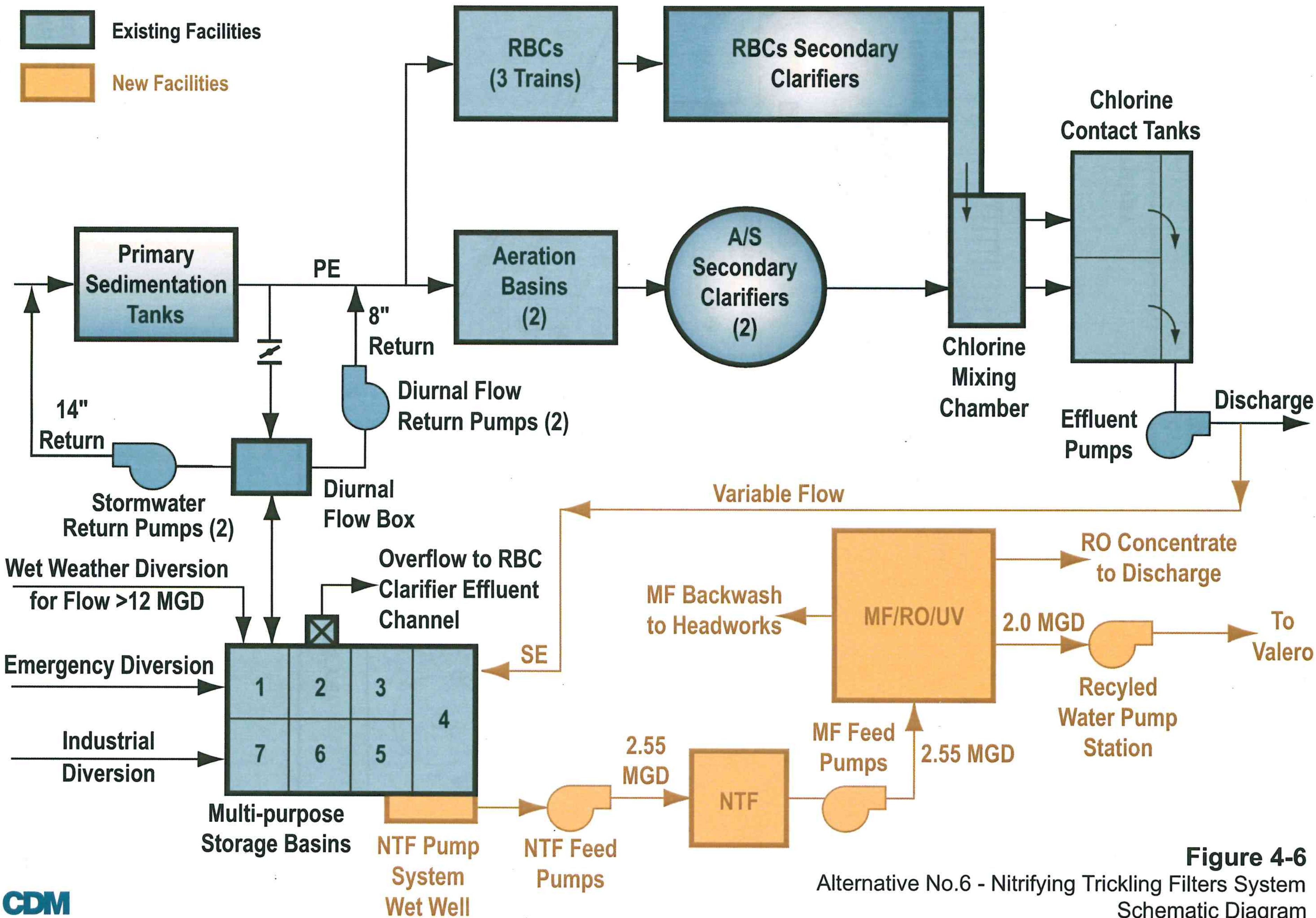

Figure 4-6
Alternative No.6 - Nitrifying Trickling Filters System Schematic Diagram

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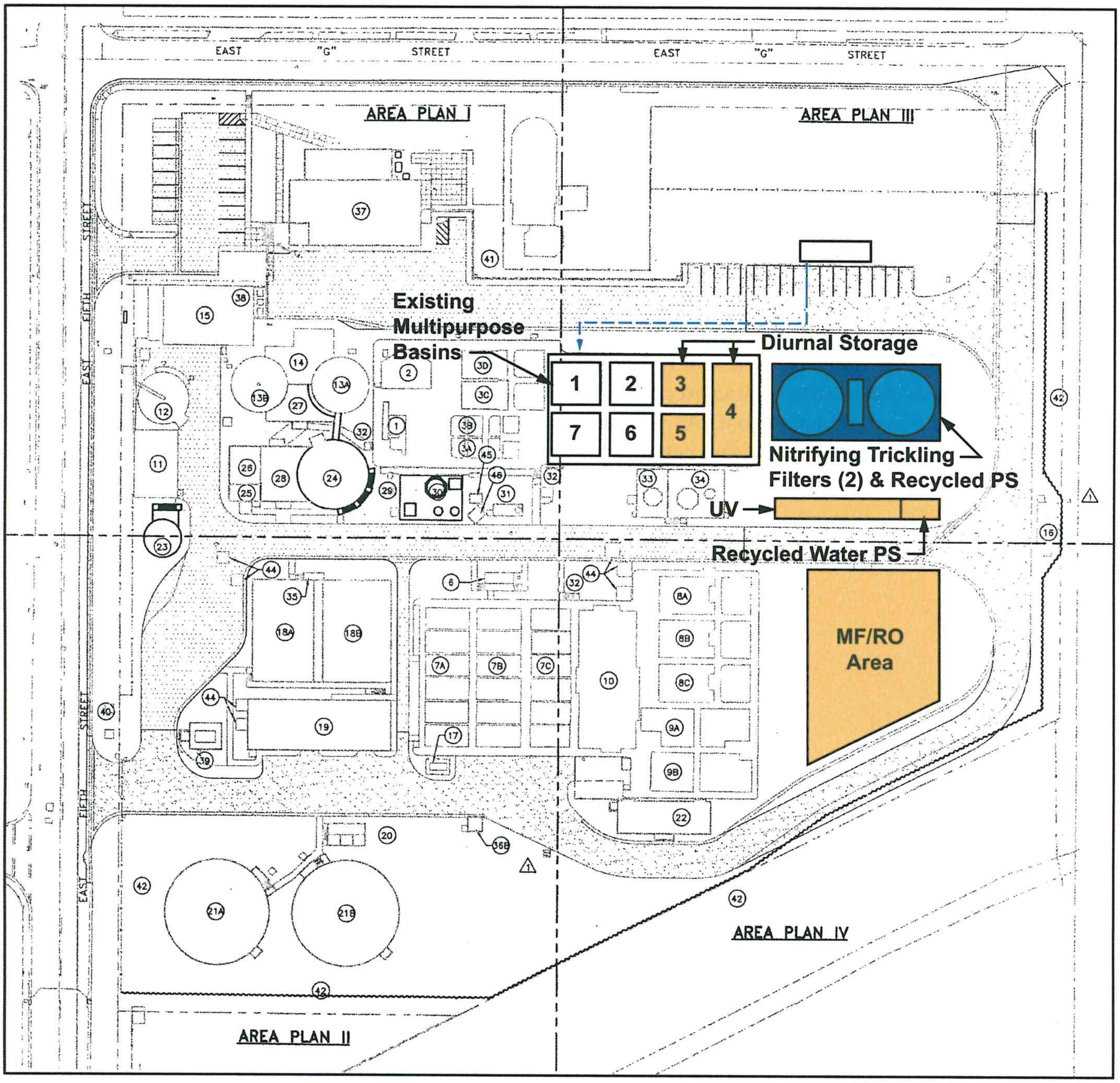

Figure 4-6A
Alternative No.6 Conceptual Site Plan
Nitrifying TF's

CDM

5.0 Estimated Construction Costs of Biological Nitrification Alternatives

Based on the conceptual designs presented in Section 4 above, additional details were developed and construction cost estimates were prepared for each of the six alternatives. For the three stand-alone alternatives, Alternative Nos. 4, 5, and 6, manufacturers were contacted for budgetary estimates for the respective equipment. We used unit prices for various components and surcharges for electrical and instrumentation and control systems that were similar to those used in the other TM's.

Table 5-1 through **Table 5-6** contain the estimated construction costs for the six alternatives. A review of the construction estimates shows that Alternative No. 4 BAF's has the highest estimated cost, while Alternative No. 1 NAS (2 AB's & 3 SC's) has the lowest estimated cost. As will be described below, however, each alternative has different reliability and capacity.

Table 5-1
Alternative No. 1 - Nitrifying Activated Sludge (2 AB's & 3 SC's)

Project Components	***Estimated Quantities***	***Units***	***Unit Costs***	***Extensions $'s***
Secondary Clarifier: 70 ft Dia x 14 ft SWD				
Structural/Concrete				
Slabs on grade	310	cy	$250	$77,500
Walls	140	cy	$500	$70,000
Effluent channel	50	cy	$700	$35,000
Piles: No Piles x L = total length. # =76	6,840	lf	$40	$273,600
Equipment/Clarifier Mechanism	1	each	$100,000	$100,000
Install Clarifier Mechanism	1	ls	$30,000	$30,000
RAS Pump	1	each	$30,000	$30,000
Piping	1	ls	$20,000	$20,000
Misc. Metals	1	ls	$15,000	$15,000
Excavation, Haul and Dispose	3,200	cy	$20	$64,000
Imported Backfill in place	400	cy	$30	$12,000
Dewatering	1	ls	$30,000	$30,000
		Subtotal-Secondary Clarifier		$757,100
Blower Building: 3 New Blowers				
Remove Existing Blowers and piping	1	ls	$15,000	$15,000
Blowers	3	each	$60,000	$180,000
Piping & Valves	1	ls	$50,000	$50,000
Misc. Metals	1	ls	$10,000	$10,000
		Subtotal-Blower Building		$255,000
Caustic Feed System				
Feed pumps, storage & containment	1	ls	$20,000	$20,000
Piping & Valves	1	ls	$15,000	$15,000
		Subtotal-Caustic Feed System		$35,000
Civil Site Work				
Subtotal of Structural, excluding piles	$182,500			
Percent of structural, excluding piles	20%			$36,500
Electrical/I&C				
Subtotal Power-Driven Mech Equipment	$330,000			
Percent of Power-Driven Mech Equipment	60%			$198,000
			Subtotal	$1,245,100
Contingency	25%			$311,300
			Subtotal	$1,556,400
Contractor's OH & P	15%			$233,500
Total Estimated Construction Cost				**$1,789,900**

Table 5-2
Alternative No. 2 - Nitrifying Activated Sludge (3 AB's & 3 SC's)

Project Components	Estimated Quantities	Units	Unit Costs	Extensions $'s
Demolition of RBC				
Remove 1 train of equipment & covers	ls	--	--	$20,000
Demolish 1 train of concrete basins	740	cy(a)	$100	$74,000
(a) Demolish, haul and dispose				
	Subtotal-Demolition of 1 RBC Train			$94,000
Aeration Tank: 40 ft w x 66 ft l x 18 ft SWD				
Structural/Concrete				
Slabs on grade	350	cy	$250	$87,500
Walls	330	cy	$500	$165,000
Elevated Slabs and small channels	70	cy	$700	$49,000
Piles: No Piles x L = total length. # = 80	7,200	lf	$40	$288,000
Diffusers	1,000	each	$50	$50,000
Air Piping		ls	$30,000	$30,000
Misc Piping		ls	$20,000	$20,000
Aluminum Covers	1200	sf	$50	$60,000
Aluminum Handrail	300	lf	$60	$18,000
Grating	400	sf	$40	$16,000
Weir Gates (manual)	3	each	$8,000	$24,000
Excavation, Haul and Dispose	2,200	cy	$20	$44,000
Imported Backfill in place	450	cy	$30	$13,500
Dewatering	1	ls	$30,000	$30,000
	Subtotal-Activated Sludge Tank			$895,000
Secondary Clarifier: 70 ft dia x 14 ft SWD				
Structural/Concrete				
Slabs on grade	310	cy	$250	$77,500
Walls	140	cy	$500	$70,000
Effluent channel	50	cy	$700	$35,000
Piles: No Piles x L = total length. # =76	6,840	lf	$40	$273,600
Equipment/Clarifier Mechanism	1	each	$100,000	$100,000
Install Clarifier Mechanism	1	ls	$30,000	$30,000
RAS Pumps	1	each	$30,000	$30,000
Piping	1	ls	$20,000	$20,000
Misc. Metals	1	ls	$15,000	$15,000
Excavation, Haul and Dispose	3,200	cy	$20	$64,000
Imported Backfill in place	400	cy	$30	$12,000
Dewatering	1	ls	$30,000	$30,000
	Subtotal-Secondary Clarifier			$757,100

Table 5-2 (continued)
Alternative No. 2 - Nitrifying Activated Sludge (3 AB's & 3 SC's)

Project Components	Estimated Quantities	Units	Unit Costs	Extensions $'s
Blower Building				
Remove Existing Blowers and piping	1	ls	$15,000	$15,000
Blowers (3,000 scfm ea)	3	each	$60,000	$180,000
Piping & Valves	1	ls	$50,000	$50,000
Misc. Metals	1	ls	$10,000	$10,000
			Subtotal-Blower Building	$255,000
Caustic Feed System				
Feed pumps, storage & containment	1	ls	$20,000	$20,000
Piping & Valves	1	ls	$15,000	$15,000
Electrical and I&C	50%		$20,000	$10,000
			Subtotal-Caustic Feed System	$45,000
			Subtotal	$2,046,100
Civil Site Work				
Subtotal of Structural, excluding piles	$484,000			
Percent of structural, excluding piles	15%			$72,600
Electrical/I&C				
Subtotal Power-Driven Mech Equipment	$310,000			
Percent of Power-Driven Mech Equipment	60%			$186,000
			Subtotal	$2,304,700
Contingency	25%			$576,200
			Subtotal	$2,880,900
Contractor's OH & P	15%			$432,100
Total Estimated Construction Cost				**$3,313,000**

Table 5-3
Alternative No. 3 - Nitrifying Activated Sludge with CEPT

Project Components	Estimated Quantities	Units	Unit Costs	Extensions $'s
Secondary Clarifier: 70 ft dia x 14 ft SWD				
Structural/Concrete				
Slabs on grade	310	cy	$250	$77,500
Walls	140	cy	$500	$70,000
Effluent channel	50	cy	$700	$35,000
Piles: No Piles x L = total length. # =76	6,840	lf	$40	$273,600
Equipment/Clarifier Mechanism.	1	each	$100,000	$100,000
Install Clarifier Mechanism	1	ls	$30,000	$30,000
RAS Pumps	1	each	$30,000	$30,000
Piping	1	ls	$20,000	$20,000
Misc. Metals	1	ls	$15,000	$15,000
Excavation, Haul and Dispose	3,200	cy	$20	$64,000
Imported Backfill in place	400	cy	$30	$12,000
Dewatering	1	ls	$30,000	$30,000
		Subtotal-Secondary Clarifier		$757,100
Blower Building				
Remove Existing Blowers and piping	1	ls	$15,000	$15,000
Blowers	3	each	$60,000	$180,000
Piping & Valves	1	ls	$50,000	$50,000
Misc. Metals	1	ls	$10,000	$10,000
		Subtotal-Blower Building		$255,000
Chemical Storage and Feed System				
Concrete Pad, Spill Containment & Canopy	1	ls	$30,000	$30,000
Ferric Chloride Storage Tank-2,000 gal	1	ls	$10,000	$10,000
Ferric Chloride Feed Pumps	2	each	$10,000	$20,000
Ferric Chloride Piping	1	ls	$15,000	$15,000
Polymer Feed Packaged Pump Systems	2	each	$15,000	$30,000
Polymer Piping, Valves & Fittings	1	ls	$15,000	$15,000
		Subtotal-Chemical Systems		$120,000
Caustic Feed System				
Feed pumps, storage & containment	1	ls	$20,000	$20,000
Piping & Valves	1	ls	$15,000	$15,000
Electrical and I&C	50%		$20,000	$10,000
		Subtotal-Caustic Feed System		$45,000
Civil Site Work				
Subtotal of Structural, excluding piles	$212,500			
Percent of structural, excluding piles	15%			$31,875
Electrical/I&C				
Subtotal Power-Driven Mech Equipment	$360,000			
Percent of Power-Driven Mech Equipment	60%			$216,000
			Subtotal	$1,627,850
Contingency	25%			$407,000
			Subtotal	$2,034,850
Contractor's OH & P	15%			$305,200
Total Estimated Construction Cost				**$2,340,050**

Table 5-4
Alternative No. 4 - Nitrifying Biological Active Filters (BAF's)

Project Components	Estimated Quantities	Units	Unit Costs	Extensions $'s
Structural & Civil				
Structural				
2 filters @ 430 sf each	860	sf	$400	$344,000
BAF Feed Pump Station (2.75 mgd)	19,000	gal	$2	$38,000
Civil Work, 10% of Structural, less piles	382,000		15%	$57,300
Pile Foundation				
Total length of piles for BAF's	1050	ft	$40	$42,000
Total length of piles for BAF PS	420	ft	$40	$16,800
			Subtotal Structural & Civil	$498,100
In-Line Screens				
Screens	2	ea	$35,000	$70,000
Electrical and I&C, % of Screens	50%		$70,000	$35,000
Piping and Valves	1	ls	$30,000	$30,000
			Sum-In-Line Screens	$135,000
BAF Feed Pumps				
Pump, Motor & VFD 20 hp	2	ea	$20,000	$40,000
Electrical and I&C, % of Pumps	75%			$30,000
			Subtotal BAF Feed Pumps	$70,000
BAFs				
Filter Equipment, includes media, BW pumps, process & BW blowers, valves & controls				
Mfgr Quote	lot	1	ls	$1,200,000
Sales Tax	%		8.25%	$99,000
Installation (% of equipment cost)	%		20%	$240,000
Process Piping	lot	ls	$30,000	$30,000
Electrical Driven Mechanical Equip	% of Total Package	35%	$420,000	
Electrical and I&C, % of Mech Equip	60%		$420,000	$252,000
			Subtotal Filters & Equipment	$1,821,000
Caustic Feed System				
Feed pumps, storage & containment	1	ls	$20,000	$20,000
Piping & Valves	1	ls	$15,000	$15,000
Electrical and I&C, % of Mech Equip	50%		$20,000	$10,000
			Subtotal-Caustic Feed System	$45,000
Building				
Blower & Pump Building	900	sf	$150	$135,000
Civil Work Associated w/ Bldg	20%		$135,000	$27,000
			Subtotal Building	$162,000
			Subtotal	$2,731,100
Add Contingency (Not including quoted equipment)			25%	$331,000
Add Contingency on equipment			10%	$131,000
Subtotal				$3,193,100
Add 15% Contractor OH & P			15%	$478,965
			Total Estimated Construction Cost	**$3,672,000**

Table 5-5
Alternative No. 5 - Tertiary Submerged Fixed-Film Nitrification

Project Components	Estimated Quantities	Units	Unit Costs	Extensions $'s
Nitrification Aeration Tank: 2 Cells 50 ft x 50 ft x 15 ft SWD)				
Civil & Structural				
Concrete				
Slabs on grade	400	cy	$250	$100,000
Walls	350	cy	$500	$175,000
Elevated Slabs and small channels	40	cy	$700	$28,000
Subtotal-Concrete				$303,000
Civil				
Civil Site Work, % of structural	20%	%	$303,000	$60,600
Excavation, Haul and Dispose	4,200	cy	$20	$84,000
Imported Backfill in place	500	cy	$30	$15,000
Dewatering	lot	ls	$20,000	$20,000
Subtotal-Civil				$179,600
Pile Foundation				
Total length of piles	5,390	lf	$40	$215,600
Misc Metals				
Aluminum Handrail	300	lf	$60	$18,000
Aluminum Covers	1,200	sf	$50	$60,000
Subtotal-Misc Metals				$78,000
Subtotal-Structural/Civil for Nitrification Aeration Tank				$776,200
Secondary Effluent Transfer Pumps				
Pump, Motor & VFD 7.5 hp	2	ea	$12,000	$24,000
Electrical and I&C, % of Pumps	75%			$18,000
Piping and Valves	1	lot	$20,000	$20,000
		Subtotal SE Transfer Pumps		$62,000
Kaldnes or IDI/Hydroxyl				
Media, diffusers & strainers (mfgr quote)	1	ls	$550,000	$550,000
Sales Tax @	8.250%			$45,400
Installation, % of Equipment	30%			$165,000
			Subtotal-Kaldnes	$760,400
Caustic Feed System				
Feed pumps, storage & containment	1	ls	$20,000	$20,000
Piping & Valves	1	ls	$15,000	$15,000
Electrical and I&C, % of Mech Equip	50%		$20,000	$10,000
		Subtotal-Caustic Feed System		$45,000
In-Line Screens				
Screens	2	ea	$35,000	$70,000
Electrical and I&C, % of Screens	50%		$70,000	$35,000
Piping and Valves	1	ls	$30,000	$30,000
		Subtotal -In-Line Screens		$135,000
Nitrification Air Supply				
2 Blowers, ea @ 3,000 scfm @ 8.1 psig	2	each	$60,000	$120,000
Air Piping & Valves	1	ls	$30,000	$30,000
Electrical and I&C, % of Blowers	60%		$120,000	$72,000
Blower Building -	600	sf	$150	$90,000
Civil Work Associated w/ Blower Bldg	20%		$90,000	$18,000
			Subtotal-Air Supply	$330,000
		Subtotal without Kaldnes		$1,348,200
Contingency, not including Kaldnes	25%			$337,100
Contingency on Kaldnes Equipment	10%			$55,000
			Kaldnes	$760,400
		Subtotal with Kaldnes		$2,500,700
Contractor's OH & P	15%			$375,100
Total Estimated Construction Cost				**$2,875,800**

Table 5-6
Alternative No. 6 - Nitrifying Trickling Filters

Project Components	Estimated Quantities	Units	Unit Costs	Extensions $'s
Trickling Filters: 2 Units, 42 ft dia x 12 ft media depth				
Civil & Structural				
Concrete				
Bottom Slabs on grade	400	cy	$250	$100,000
Tilt-Up Walls	11,080	sf	$30	$332,400
Plenum perimeter walls	44	cy	$700	$30,800
recycle pump station wet well	8000	gal	$4	$32,000
piers and precast beams -support system	2	ls	$15,000	$30,000
Subtotal-Concrete				$525,200
Civil				
Civil Site Work, % of structural	20%	%	$525,200	$105,040
Excavation, Haul and Dispose	2,000	cy	$20	$40,000
Imported Backfill in place	390	cy	$30	$11,700
Dewatering	lot	ls	$20,000	$20,000
Subtotal-Civil				$176,740
Pile Foundation				
Total length of piles	3,000	lf	$40	$120,000
Subtotal-Structural/Civil for Trickling Filters & recycle PS				$821,940
Combination Feed & Recycle Pumps				
Combined TF Feed & Pump, Motor & VFD 10hp	3	ea	$15,000	$45,000
Electrical and I&C, % of Pumps	75%			$33,750
Piping and Valves	1	lot	$20,000	$20,000
Subtotal Recycle Pumps				$98,750
Secondary Effluent Transfer Pumps				
Pump, Motor & VFD 7.5 hp	3	ea	$12,000	$24,000
Electrical and I&C, % of Pumps	75%			$18,000
Piping and Valves	1	lot	$20,000	$20,000
Subtotal SE Transfer Pumps				$62,000
Plastic Media				
Media	34,000	cf	$6	$204,000
Media Installation	34,000	cf	$1	$34,000
Subtotal-Media				$238,000
Caustic Feed System				
Feed pumps, storage & containment	1	ls	$20,000	$20,000
Piping & Valves	1	ls	$15,000	$15,000
Electrical and I&C, % of Mech Equip	50%		$20,000	$10,000
Subtotal-Caustic Feed System				$45,000
Effluent Screen				
Screen & auger	1	ea	$35,000	$35,000
Electrical and I&C, % of Screens	50%		$35,000	$17,500
Subtotal -In-Line Screens				$52,500
TF Induced Air Supply				
8 fans, ea @ 1,500 scfm @ 2-in water, including sound enclosures	8	each	$5,500	$44,000
Sales Tax @	8.250%		$44,000	$3,600
Air Piping & Valves	8	ls	$5,000	$40,000
Electrical and I&C, % of Fans & Motors	60%		$44,000	$26,400
Subtotal-Air Supply				$114,000
			Subtotal	$1,432,200
Contingency	25%			$358,100
			Subtotal	$1,790,300
Contractor's OH & P	15%			$268,500
Total Estimated Construction Cost				**$2,058,800**

6.0 Estimated Operating & Maintenance Costs of Biological Nitrification Alternatives

Operating requirements, including power, labor, chemicals and other consumables were estimated for each of the six alternatives. Power was estimated at $0.12 per kilowatt hour (kWhr); labor at $50 per hour, including City administrative overhead; ferric chloride at $0.30 per gallon at 40% strength; and, polymer at $1.00 per pound. For Alternative Nos. 1, 2 and 3, which are dependent on the total flow to the entire WWTP, an annual average flow over the 20-year planning period was assumed at 3.8 mgd. For Alternative Nos. 4, 5, and 6, a constant flow of 2.55 mgd (as required input flow to the MF/RO system) over the 20-year period was assumed. Also, for Alternative Nos. 4 and 5, the manufacturers provided media replacement costs.

All alternatives require the addition of alkalinity, because the nitrification process consumes alkalinity as it converts ammonia to nitrate. Based on plant data of 190 mg/L alkalinity (as $CaCO_3$), we estimate that an equivalent amount of 60 mg/L of alkalinity must be added to insure adequate chemical balance in the process. For this TM it has been assumed that alkalinity would be added in the form of caustic soda, although other chemical will be evaluated in the design phase. The cost of caustic was assumed at $0.30 per equivalent pound.

6.1 Estimated O&M Costs of Alternative No. 1 NAS (2 AB's & 3 SC's)

Estimates of power for additional process air were made along with other O&M requirements, which are detailed in **Table 6-1**.

Using the estimated O&M requirements shown in **Table 6-1**, annual O&M cost estimates were made using the unit prices stated above. **Table 6-2** presents the estimated O&M costs for Alternative No. 1.

6.2 Estimated O&M Costs for Alternative No. 2 NAS (3AB's & 3 SC's)

Estimates of power for additional process air were made along with other O&M requirements, which are detailed in **Table 6-3**.

Using the estimated O&M requirements shown in **Table 6-3**, annual O&M cost estimates were made using the unit prices stated above. **Table 6-4** presents the estimated O&M costs for Alternative No. 2.

6.3 Estimated O&M Costs of Alternative No. 3 NAS & CEPT

Estimates of power for additional process air were made along with chemicals and other O&M requirements, which are detailed in **Table 6-5**.

Using the estimated O&M requirements shown in **Table 6-5**, annual O&M cost estimates were made using the unit prices stated above. **Table 6-6** presents the estimated O&M costs for Alternative No. 1.

Table 6-1
Estimated O&M Requirements for Alternative No. 1 - Nitrifying Activated Sludge (2 AB's & 3 SC's)

O&M Cost Items		Units	Estimated Quantities
Power Consumption			
	Blowers for Increased Air to Nitrify	kWhr/yr	294,100
	RAS Pumping	kWhr/yr	63,700
Total Estimated Additional Power		kWhr/yr	357,800
Chemicals			
	Caustic for Alkalinity Replacement	lb/yr	578,400
Increased Labor – O&M		hr/yr	150

Table 6-2
Estimated O&M Costs for Alternative No. 1 - NAS (2 AB's & 2 SC's)

O&M Cost Items		Units	Estimated Quantities	Unit Costs	Unit Cost Per	Annual Extensions $1,000/yr
Power Consumption		kWhr/yr	357,800	$0.12	kWhr	$43
Chemicals						
	Caustic	lb/yr	578,400	$0.20	lb	$116
Labor – Operations & Maintenance		hr/yr	150	$50	hr	$8
Mechanical & Electrical Equipment Repair & Replacement		$	$496,000	2%	yr	$10
					Subtotal	$176
					Contingency at 15%	$26
Total Estimated Annual O&M Cost						**$202**

Table 6-3
Estimated O&M Requirements for Alternative No. 2 - Nitrifying Activated Sludge (3 AB's & 3 SC's)

O&M Cost Items		Units	Estimated Quantities
Power Consumption			
	Blowers for Increased Air	kWhr/yr	318,600
	RAS Pumping	kWhr/yr	78,400
Total Power Consumption		kWhr/yr	397,000
Chemicals			
	Caustic for Alkalinity Replacement	lb/yr	578,400
Additional Labor for O&M		hr/yr	150

Table 6-4
Estimated O&M Costs for Alternative No. 2 - Nitrifying Activated Sludge (3 AB's & 3 SC's)

O&M Cost Items		Units	Estimated Quantities	Unit Costs	Unit Cost Per	Annual Extensions $1,000/yr
Power Consumption		kWhr/yr	397,000	$0.12	kWhr	$48
Chemicals						
	Caustic	lb/yr	578,400	$0.20	lb	$116
Labor – Operations & Maintenance		hr/yr	150	$50	hr	$8
Mechanical & Electrical Equipment Repair & Replacement		$	$620,000	2%	yr	$12
					Subtotal	$184
					Contingency at 15%	$27
Total Estimated Annual O&M Cost						**$211**

Table 6-5
Estimated O&M Requirements for Alternative No. 3 - Nitrifying Activated Sludge with CEPT

O&M Cost Items	Units	Estimated Quantities
Power Consumption		
Blowers for Increased Air	kWhr/yr	245,100
RAS Pumping	kWhr/yr	63,700
Total Power Consumption	kWhr/yr	308,800
Chemicals		
Ferric Chloride (20 mg/L)	lb/yr	219,200
Caustic for Alkalinity Replacement	lb/yr	578,400
Polymer	lb/yr	11,000
Additional Labor for O&M	hr/yr	350

Table 6-6
Estimated O&M Costs for Alternative No. 3 - Nitrifying Activated Sludge with CEPT

O&M Cost Items	Units	Estimated Quantities	Unit Costs	Unit Cost Per	Annual Extensions $1,000/yr
Power Consumption	kWhr/yr	308,800	$0.12	kWhr	$37
Chemicals					
Ferric Chloride	lb/yr	219,200	$0.35	lb	$77
Caustic for Alkalinity Replacement	lb/yr	578,400	$0.20	lb	$116
Polymer	lb/yr	11,000	$1.00	lb	$11
Subtotal Chemicals					$203
Additional Labor for O&M	hr/yr	350	$50	hr	$18
Mechanical & Electrical Equipment Repair & Replacement	$	$770,000	2%	yr	$15
				Subtotal	$273
				Contingency at 15%	$41
Total Estimated Annual O&M Cost					**$314**

6.4 Estimated O&M Costs of Alternative No. 4 Nitrifying BAF's

Estimates of power for additional process air were made along with other O&M requirements, which are detailed in **Table 6-7**.

Using the estimated O&M requirements shown in **Table 6-7**, annual O&M cost estimates were made using the unit prices stated above. **Table 6-8** presents the estimated O&M costs for Alternative No. 4.

6.5 Estimated O&M Costs of Alternative No. 5 TSFF Nitrification

Estimates of power for additional process air were made along with other O&M requirements, which are detailed in **Table 6-9**.

Using the estimated O&M requirements shown in **Table 6-9**, annual O&M cost estimates were made using the unit prices stated above. **Table 6-10** presents the estimated O&M costs for Alternative No. 5.

6.6 Estimated O&M Costs of Alternative No. 6 Tertiary Nitrifying Trickling Filters

Estimates of power for additional process air were made along with other O&M requirements, which are detailed in **Table 6-11**.

Using the estimated O&M requirements shown in **Table 6-11**, annual O&M cost estimates were made using the unit prices stated above. **Table 6-12** presents the estimated O&M costs for Alternative No. 6.

Table 6-7
Estimated O&M Costs for Alternative No. 4 - Nitrifying Biological Active Filters

O&M Cost Items	Units	Estimated Quantities
Power Consumption		
Process Air Compressors	kWhr/yr	294,000
Backwash Air Compressors	kWhr/yr	24,500
Backwash Pumps	kWhr/yr	26,100
Total Power Consumption	kWhr/yr	344,600
Chemicals		
Caustic for Alkalinity Replacement	lb/yr	395,700
Filter Media Replacement	ton/yr	3
Labor – Operations & Maintenance	hr/yr	800
Waste Backwash Water Treatment	mg/yr	54

Table 6-8
Estimated O&M Costs for Alternative No. 4 - Nitrifying Biological Active Filters

O&M Cost Items	Units	Estimated Quantities	Unit Costs	Unit Cost Per	Annual Extensions $1,000/yr
Power Consumption	kWhr/yr	344,600	$0.12	kWhr	$41
Chemicals					
Caustic for Alkalinity Replacement	lb/yr	395,700	$0.20	lb	$79
Filter Media Replacement	ton/yr	3	$188	ton	$1
Labor – Operations & Maintenance	hr/yr	800	$50	hr	$40
Mechanical & Electrical Equipment Repair & Replacement	$	$1,658,800	2%	yr	$33
Waste Backwash Water Treatment	mg/yr	54	$300	mg	$16
				Subtotal	$210
				Contingency at 15%	$32
Total Estimated Annual O&M Cost					**$242**

Table 6-9
Estimated O&M Requirements for Alternative No. 5 – Tertiary Submerged Fixed-Film Nitrification

O&M Cost Items	Units	Estimated Quantities
Power Consumption	kWhr/yr	313,600
Chemicals		
Caustic for Alkalinity Replacement	lb/yr	395,700
Media Replacement	cf/yr	190
Labor – Operations & Maintenance	hr/yr	500

Table 6-10
Estimated O&M Costs for Alternative No. 5 - TSFF Nitrification

O&M Cost Items	Units	Estimated Quantities	Unit Costs	Unit Cost Per	Annual Extensions $1,000/yr
Power Consumption	kWhr/yr	313,600	$0.12	kWhr	$38
Chemicals					
Caustic for Alkalinity Replacement	lb/yr	395,700	$0.20	lb	$79
Media Replacement	cf/yr	190	$80	cf	$15
Labor – Operations & Maintenance	hr/yr	500	$50	hr	$25
Mechanical & Electrical Equipment Repair & Replacement	$	$501,250	2%	yr	$10
				Subtotal	$167
				Contingency at 15%	$25
Total Estimated Annual O&M Cost					**$192**

Table 6-11
Estimated O&M Requirements for Alternative No. 6 - Nitrifying Trickling Filters

O&M Cost Items	Units	Estimated Quantities
Power Consumption	kWhr/yr	265,200
Chemicals		
Caustic for Alkalinity Replacement	lb/yr	395,700
Labor – Operations & Maintenance	hr/yr	500

Table 6-12
Estimated O&M Costs for Alternative No. 6 - Nitrifying Trickling Filters

O&M Cost Items	Units	Estimated Quantities	Unit Costs	Unit Cost Per	Annual Extensions $1,000/yr
Power Consumption	kWhr/yr	265,200	$0.12	kWhr	$32
Chemicals					
Caustic for Alkalinity Replacement	lb/yr	395,700	$0.20	lb	$79
Labor – Operations & Maintenance	hr/yr	500	$50	hr	$25
Mechanical & Electrical Equipment Repair & Replacement	$	$345,375	2%	yr	$7
				Subtotal	$143
				Contingency at 15%	$22
Total Estimated Annual O&M Cost					**$165**

7.0 Quantitative Evaluation of Alternatives

7.1 Capital Cost Estimates

The capital cost of a project includes both the initial construction cost plus engineering and construction management costs required to implement the project. An amount of 35% of the estimated construction cost has been added to account for these costs.

7.2 Life Cycle Cost Analysis

The capital and annual O&M cost estimates presented herein are for comparative purposes only. These cost estimates are used to determine the biological nitrification alternative that is the most cost-effective in relation to the other alternatives. A more detailed construction cost estimate will be developed for the selected alternative as part of the preliminary design.

Using estimated capital and annual O&M costs for each alternative system, present worth values were developed to compare the life-cycle costs of the six alternatives. Present worth is defined as that amount of money it takes to fund the capital investment of a project, as well as its annual operating and maintenance costs, over a period of time, given the cost of money (interest) during the evaluation period. For this analysis, the time period used was 20 years and the interest rate was six percent. **Table 7-1** presents the results of this analysis.

As can be seen from inspection of **Table 7-1**, Alternative No. 6 has the lowest present worth value among the six alternatives analyzed. Alternative No. 1 has the next lowest present worth value by approximately 1.5%. Although Alternative No. 1 has the lowest estimated capital cost, however, as discussed above in Section 3 and below in Section 8, there are significant reliability limitations regarding Alternative No. 1, in that it cannot consistently meet the maximum secondary effluent ammonia criterion of 2 mg/L.

Table 7-1
Summary of Economic Analysis of Biological Nitrification Alternatives

Component	***Alt No. 1 NAS (2&3) $1,000's***	***Alt No. 2 NAS (3&3) $1,000's***	***Alt No. 3 NAS&CEPT $1,000's***	***Alt No. 4 BAFs $1,000's***	***Alt No. 5 TSFF $1,000's***	***Alt No. 6 TNTF $1,000's***
Estimated Construction Costs (1)	$1,790	$3,310	$2,340	$4,580	$2,880	$2,060
Add 35% for Engineering and CM	$630	$1,160	$820	$1,600	$1,010	$720
Total Estimated Capital Cost	$2,420	$4,470	$3,160	$6,180	$3,890	$2,780
Estimated Annual O&M Costs (2)	$202	$211	$314	$242	$192	$165
Present Worth of O&M Costs (3)	$2,320	$2,420	$3,610	$2,780	$2,200	$1,890
Total Estimated Present Worth Values	**$4,740**	**$6,890**	**$6,770**	**$8,930**	**$6,090**	**$4,670**

(1) From Tables 5-1 through 5-6
(2) From Tables 6-2, 6-4, 6-6, 6-8, 6-10 & 6-12
(3) PWF: i = 6% and n = 20 yrs

8.0 Qualitative Evaluation of Alternatives

In addition to capital cost, operating costs and overall present worth values, it is appropriate to evaluate other qualitative factors to aid in the selection of the best biological nitrification process. Below is a discussion of pertinent qualitative factors. **Table 8-1** contains a tabular summary of these discussions.

Table 8-1
Summary of Qualitative Evaluation of Nitrification Alternatives

Qualitative Factors	*Alt No. 1*	*Alt No. 2*	*Alt No. 3*	*Alt No. 4*	*Alt No. 5*	*Alt No. 6*
Impact on Existing Facilities	Moderate	High	Moderate	Low	Low	Low
Ease of Operation	Good	Good	Moderate	Moderate	Good	Good
Ease of Implementation	Moderate	Difficult	Moderate	Good	Good	Good
Incremental Expandability	Difficult	Difficult	Difficult	Moderate	Moderate	Moderate
Equipment Reliability	Good	Good	Good	Good	Good	Good
Process Reliability	Limited	Good	Limited	Good	Limited	Good
Proven Technology	Good	Good	Good	Good	Limited	Good
Process Complexity	Moderate	Moderate	Moderate	High	Low	Moderate
Power Demand	High	High	High	Moderate	Low	Lowest
Visual Impact	Low	Low	Low	High	Low	Moderate

Water Reuse System Reliability: Prior to presenting a qualitative evaluation of the alternatives, it is appropriate to revisit staff proposed reliability features of the water reuse system components. The Water Reuse Treatment Project is not being designed to provide ammonia free recycled water at a continuous 2 mgd (or a reduced design capacity based on financial constraints) flow rate on a 24 hour, 7-day per week basis. In TM-4, it was determined that stand-by power would not be provided to the Water Reuse Project. It was agreed that during power outages or reduced production events that potable water could be used for make-up supply. It was also agreed that systems requiring annual maintenance could be performed during the winter when raw water supplies are not at a premium.

All pumping systems will have a stand-by pumping unit and all process air supply systems will have a stand-by compressor unit. Mechanical equipment such as these are more subject to occasional failures. However, it is not cost effective to have standby process units, particularly for any of the stand-alone biological nitrification systems. Hence, each system is designed with two units, each capable of processing 50 percent of the design flow. If one unit is down for repair, production will be proportionately reduced.

Impact on Existing Facilities: Adding a third secondary clarifier and a third RAS pump have been planned from prior designs to expand the treatment facilities. Replacing the existing process blowers will be highly disruptive. For Alternative No. 2, demolishing one of the three trains of RBC's, in order to construct a 3rd AB, will be challenging and will then leave the operators with less flexibility for wet weather events. Other than change the "site usage" of the RBC train into a new aeration basin under Alternative No. 2, Alternative Nos. 1 and 2 make slight impacts on existing facilities. Alternative No. 3 will add more sludge to the anaerobic digestion system. CDM has analyzed the digesters and have determined that they can handle adequately the extra load.

Both the stand-alone system, Alternative Nos. 4 and 5 consume additional space. However, being stand alone systems, they pose little other impact to the existing plant. The BAF's of Alternative No. 4 will generate a backwash flow stream that must be recycled back through the plant for processing.

Ease of Operation: Biological nitrification is generally somewhat "touchy" to operate in that the operators must keep on top of operating parameters lest the system fall out of the nitrification mode or the bacteria become "washed out" of the process. Separate, stand-alone systems, such as BAF, TSFF, and NTFs are much less susceptible to such upsets. However, BAF's have several additional mechanical systems, including two air systems (process and backwash) and a backwash water pump system. NTFs have recycle pumps and rotary distributors which basically run at pre-set constant speed.

Ease of Implementation: All three stand-alone systems are relatively easy to implement. Adding the third aeration basin under Alternative No. 2 is the most disruptive, owing to demolition of an RBC train. Replacing the activated sludge process air blowers will be disruptive to plant operations and may require a temporary system while the blowers are being replaced. Significant coordination with plant operations during construction will be required for this change out.

Incremental Expandability: The stand-alone systems (Alternative Nos. 4, 5, and 6) can be designed for ease of expansion, should it be determined that there may be additional demand for recycled water. Or, if the initial capacity of the project is reduced, owing to initial funding limitation, they can be designed to be expanded to the full, 2 mgd capacity later.

For the activated sludge alternatives, modular expansions for the water reuse project are really not practical. The sizes of additional aeration basins and secondary clarifiers should be the same as the existing to provide sufficient nitrification capacity for the full-plant flow and to facilitate hydraulic flow split.

Equipment Reliability: All the alternatives use standard mechanical equipment, such as blowers, compressors and pumps. Each one of these would be provided with a stand-by unit. There are several automatically operated valves associated with the BAF's, which may reduce the reliability of Alternative No. 4.

Process Reliability: Process reliability, as differentiated from equipment or mechanical reliability, refers to the ability of a treatment process to consistently produce an effluent that meets design water quality requirements. An effluent ammonia concentration of 2.0 mg/L is can be routinely achieved by a NAS process. Operating requirements and limitations for NAS systems are well established, and with proper design and operation

NAS processes can be expected to consistently perform as intended. While the number of tertiary nitrifying BAF plants is significantly less than for NAS, a significant number of them are operating successfully (see **Appendix B**). Selected data from a number of operating facilities including a large BAF installation in Onondaga County New York demonstrate that BAF technology can consistently meet low effluent ammonia requirements. We were only able to identify three full-scale plants in the United States that use submerged fixed-film processes for nitrification. Based on selected data obtained for two of the three facilities, effluent ammonia concentrations are possible but not routinely achieved. More information on these facilities is needed to make a determination of the reasons for the effluent ammonia variability at the existing facilities.

Since nitrifying organisms are sensitive to many toxic compounds, an effective industrial pretreatment program is essential to keep materials out of the wastewater that could upset the nitrification process. Tertiary nitrification systems are somewhat less susceptible to upset from toxins dumped into the municipal system because the upstream, activated sludge process will attenuate, and possibly remove substances harmful to the nitrifiers.

Proven Technology: Of the six final alternatives most have long established records of performance on a world-wide basis. In particular, nitrifying activated sludge processes have been in use for many years throughout the United States, and it is a well proven technology. BAF technology has been in use for several decades but still does not have the installed facility base that activated sludge does. While there are likely thousands of municipal wastewater treatment plants using nitrifying activated sludge, as of 2001 there were somewhat over one hundred BAF installations. Still, the size and scope of the existing BAF installations are significant enough to consider BAF technology well proven for this application. Substantially more installed and operating BAF capacity exists in Europe than in North America; however, the number of facilities in the United States has increased significantly over the last ten years, and includes several facilities in California in the same application. Submerged fixed-film technology is not new, but use of the technology has been mainly limited to moving bed biofilm and IFAS applications where the media is used for secondary treatment and nutrient removal process. Use of submerged fixed-film media for tertiary nitrification of secondary effluent in the USA is very limited. NTF have been used for at least 10 years, and in some cases 20 years, for biological nitrification. Sunnyvale, California, is a good example of a plant that successfully employs NTFs for nitrification.

Process Complexity: Process complexity considers the number and types of mechanical equipment and process control systems required to operate the facility. TSFF-type processes are perhaps the simplest of the alternatives evaluated since there is only one set of pumps and process air blowers. Operators must only maintain adequate dissolved oxygen (DO) and alkalinity for the process to operate effectively. No sludge separation or recycle streams are necessary. All the NAS process alternatives are very similar to each other, and to the existing activated sludge process. Operating complexity for NAS is not much different than for secondary activated sludge but does require more attention to solids inventory, alkalinity, and DO concentrations. Although BAFs are simple from a process perspective, operations are more complex with two sets of pumps (feed and backwash) and blowers (process air and backwash air) and several automatic valves. A programmable logic controller (PLC) is desirable to automate backwashing and to minimize operator attention. With the use of standard process controls, BAFs operations can be completely automated allowing unattended operation for extended periods. NTFs have combined feed and recycle pumps which pump the water to the top of each tower where it is distributed over the media by a rotary distributor. Also, induced draft, constant speed fans provide air for the process. All these equipment items run at constant speed.

Power Demand: Power is required for pumping, process aeration and for backwashing filters. To the extent that the alternatives have different hydraulic grade requirements, pumping requirements vary. BAFs require the largest hydraulic grade differential to operate the process and thus have the highest pumping requirements. BAFs also require pumps to backwash the units at regular intervals.

Although a fixed amount of oxygen is required to convert ammonia to nitrate (about 4.6 lb O_2/lb NH_4), the mass of ammonia converted to nitrate varies among the alternatives. All three of the nitrifying activated sludge process treat the entire plant flow, and thus nitrify all the ammonia. Whereas, the tertiary alternatives are required to nitrify only the ammonia in the secondary effluent stream, necessary as input to the water reuse system. Differences in the field oxygen transfer efficiency (FOTE) for each technology also affect the power required for process air. BAFs have a FOTE of about 20 percent while fine pore aeration in NAS systems have an FOTE of about 10 to 12 percent. We have not been able to obtain results from aeration tests for the TSFF processes so we have relied on the manufacturers' estimates for this evaluation. Alternative No. 3, which adds chemicals to the primary clarifiers, has a reduced power demand because more of the BOD load and some of the nitrogen load are removed by primary treatment.

Table 8-2 contains a summary of the power requirements for each alternative, as well as the estimated annual cost of same, assuming the unit cost of power is $0.12/kWhr. As can be seen from review of the data in **Table 8-2**, Alternative No. 6 NTFs has the lowest estimated power demand and cost.

Table 8-2
Summary of Estimated Energy Demands by Biological Nitrification Alternatives

Component	Alt No. 1 NAS (2&3)	Alt No. 2 NAS (3&3)	Alt No. 3 NAS&CEPT	Alt No. 4 BAFs	Alt No. 5 TSFF	Alt No. 6 TNTF
Estimated Energy, kWhr/Yr	357,800	397,000	308,800	344,600	313,600	265,200
Estimated Energy Cost, $1,000's per Year	$43	$48	$37	$41	$38	$32

Visual Impact: Alternative No. 4 BAF's have a high physical profile with the top of the structures being of about 25 feet from grade. Neighbors to the north of the plant may be concerned about the height of the facilities. The only mitigation for this visual impact would be to bury a portion of the filters in the ground. This would increase the structural costs significantly.

9.0 Conclusions and Recommendations

Conclusions

Based on the evaluation of the alternatives presented in this Supplement to TM-1, the following conclusions can be drawn:

1. Nitrifying Activated Sludge Alternative No. 1 does not provide reliable effluent quality of 2 mg/L ammonia for current average day flow rates.

2. Providing a reliable nitrifying activated sludge system by modifying the City's activated sludge system will be highly disruptive and result in a high capital and operating cost, compared with other available, stand-alone alternatives.

3. Three stand-alone tertiary, biological nitrification alternatives are capable of meeting the 2 mg/L ammonia criterion. Biological activated filters and nitrifying trickling filters have more proven performance as stand-alone nitrification systems, than do submerged fixed film systems.

4. BAF's have a high equipment profile of about 25 feet; they also have the highest capital and operating cost.

5. Alternative No. 6 Tertiary NTF's appears to be the most cost-effective alternative that can meet the ammonia criterion of 2 mg/L.

6. The estimated capital cost of Alternative No. 6 is within the Water Reuse Project budget allocation for nitrification for a 2 mgd project, as presented in the project cost estimate update, dated 8 March 2005.

7. Using a stand-alone nitrification system will avoid operational problems at the City's basic secondary treatment system during wet weather periods when it must accommodate high flows and still meets its NPDES permit requirements.

Recommendation

Based on the evaluations conducted and the conclusions reached in the performance of this study, CDM recommends that City staff, along with CDM, visit existing treatment plants that have stand-alone NTFs as their nitrification system (such as Sunnyvale, CA) to learn their operating characteristics and performance, and then determine if they are comfortable that this type of biological nitrification will consistently meet 2 mg/L ammonia.

Appendix A
Acronyms

Appendix A
List of Acronyms

AB	aeration basin
ADWF	Average Dry Weather Flow
AF	acre-feet
AFY	acre-feet per year
AS	Activated Sludge
AWWA	American Water Works Association
BAAQMD	Bay Area Air Quality Management District
BAF	biological aerated filter
BFP	belt filter press
BNR	Biological Nutrient (Nitrogen) Removal
BOD	biochemical oxygen demand
BOD_5	5-day Biochemical Oxygen Demand
BTU	British Thermal Unit
CAA	Clean Air Act
CCR	California Code of Regulations
CDM	Camp Dresser & McKee Inc.
CEPT	chemically enhanced primary treatment
cf	cubic foot
CFR	Code of Federal Regulations
CIP	clean-in-place
COE	U.S. Army Corps of Engineers
CPI	Consumer Price Index
CT	Product of chlorine dosage and contact time
CWA	Clean Water Act
DAF	Dissolved Air Flotation Thickener
DG	Digester Gas
DL	Dockline
DO	dissolved oxygen
DOHS	State of California Department of Health Services
EDR	Electrodialysis Reversal
EHRC	enhanced high rate clarification
$ENRCCL_{SF}$	Engineering News Record Construction Cost Index of San Francisco Area
EPA	United States Environmental Protection Agency
FOTE	field oxygen transfer efficiency
FY	Fiscal year
gpd	gallons per day
gpm	gallons per minute
HDPE	High Density Polyethylene
HRT	Hydraulic Residence Time
icfm	inlet cubic feet per minute
IDI	Infilco Degremont Incorporated
IFAS	integrated fixed film activated sludge

kV	KiloVolt (1000 Volts)
kW	KiloWatt (1000 Watts)
kWhr	kilowatt hour
L	liter
MBR	membrane bioreactor
MF	microfiltration
mg	milligram
mgal	million gallons
mg/L	milligram per liter
mgd	million gallons per day
mL	milliliter
MLSS	Mixed Liquor Suspended Solids
mW	MegaWatt (1,000,000 Watts)
NAS	nitrifying activated sludge
NBA	North Bay Aqueduct
NF	nanofiltration
NPDES	National Pollutant Discharge Elimination System
NTF	nitrifying trickling filters
NTU	Nephelometric Turbidity Unit
O&M	Operation and Maintenance
OH	overhead
OSHA	Occupational Safety and Health Administration
PE	primary effluent
PLC	programmable logic controller
POTW	Publicly Owned Treatment Works
ppd	pounds per day
PS	pump station
PSM	Process Safety Management
PVC	polyvinyl chloride
PW	present worth
PWWF	peak wet weather flow
RAS	return activated sludge
RBC's	rotating biological contactors
RO	reverse osmosis
RWQCB	Regional Water Quality Control Board – San Francisco Bay Region
RWSPS	recycled water supply pump station
SC	secondary clarifier
SCADA	Supervisory Control and Data Acquisition System
sf	square feet
SPW	State Project Water
SRT	solids (biomass) retention time
Sta	Station
SWRCB	State Water Resources Control Board
TDS	Total Dissolved Solids
Therm	100,000 BTUs, equivalent to 100 cubic feet of natural gas

TIN	Total Inorganic Nitrogen (total of ammonia-nitrogen, nitrite-nitrogen, and nitrate-nitrogen)
Title 22	California Code of Regulations, Title 22 (Water Recycling Criteria)
TKN	total Kjeldahl nitrogen
TOC	Total Organic Carbon
TSFF	Tertiary Submerged Fixed Film (nitrification)
TSS	Total Suspended Solids
USDA	U.S. Department of Agriculture
UV	ultraviolet light
UVT	UV transmittance
VOC	Volatile Organic Carbon
WRTP	Water Reuse Treatment Plant
WWTP	Wastewater Treatment Plant

Appendix B
Partial List of BAF Installations

Biofor™ Installation List

	Installation	Process	Number of Filters	Filter Area (Ft^2/Cell)	Average (Peak) Flow (MGD)	Construction Start-Up
USA	Binghamton-Johnson City, NY	Biofor C	8	1400	44 (70)	2005*
		Biofor N	8	1360		
		Biofor DN	4	840		
	Breckenridge, CO	Biofor N	4	278	1.0 (2.3)	1998
	Corpus Christi, TX	Biofor C	6	314	1.8	2000
	Evesham, NJ	Biofor N	3	192	1.7	1997
	Irvine Ranch, CA	Biofor DN	2	60	1.3	1998
	Neptune, NJ	Biofor N	4	1131	8.5 (11)	2003
	Roanoke, VA	Biofor C	6	1036	14	1998
		Biofor N	6	649		
	West Basin, CA MWD for Arco	Biofor N	1	315	0.9 (1.1)	1999
	West Basin, CA MWD for Chevron	Biofor N	4	315	5	1995
	West Basin, CA MWD for Mobil	Biofor N	4	315	5	1995
	West Warwick, RI	Biofor N	4	1080	10.5 (25.34)	2004*
		Biofor DN	4	448		
	Wetzel Rd, NY	Biofor C	4	540	7.9 (15.9)	2005*
		Biofor N	4	434		
France	Acheres	Biofor N	1	1119	7.1 (9.1)	1989
	Ahlstrom Sibille	Biofor C	6	339	3.8 (5.1)	1994
	Allos	Biofor C	4	152	0.6 (1.9)	1990
	Annecy Station Siloe	Biofor C	6	897	7.9 (16.5)	1997
		Biofor N	12	1122		
	Annemasse	Biofor C	10	786	6.3 (22.2)	1997
	Arjo Wiggins	Biofor C	4	348	4.1	1996
	Arjobex - Industrial	Biofor CN	2	248	0.0	1999
	Beaufort Sur Doron	Biofor C	4	220	3.0 (10.5)	2000
		Biofor N	3	220		
	Bordeaux	Biofor CN	6	786	11.4	1993
	Bordeaux Station Clos de Hilde	Biofor C	4	786	10.3	1994
	Bouc Bel Air	Biofor C	4	188	1.2 (2.0)	1987
	Bourg D'Oisans	Biofor CN	6	366	3.4 (7.9)	1992
	Brest Maison Blanche	Biofor C	6	313	8.6	2000
	Champsaur	Biofor C	4	151	0.8 (1.8)	1992
	Charles Des Gaulle Airport	Biofor C	2	560	4.5 (10.9)	2000
	Colombes - Seine Center	Biofor C	12	1119	63.4 (274)	1998
		Biofor CN	12	1119		
		Biofor DN	12	1119		
	Corbeil	Biofor C	7	754	4.0 (8.0)	1991
	Elf Atochem	Biofor C	2	86	0.4	1999
	Etretat	Biofor C	2	114	1.3 (0.3)	1993
	Fontaine sur Saone	Biofor C	4	441	2.6 (7.6)	1992
	Ghisonaccia et Prunelli Di Fiumorbo	Biofor CN	4	220	0.6 (1.5)	1994
	Grenoble	Biofor CN	14	754	38.8 (45.7)	1991
	Greoux les Bains	Biofor C	4	151	1.1 (2.5)	1988
	Guerimand-Voiron Paper	Biofor C	4	248	2.2	1992
	Guillestre	Biofor C	4	151	0.7 (1.9)	1992
	La Pointe Des Negres	Biofor CN	4	312	2.7	1999
	Marseille	Biofor C	2	67	0.7 (1.9)	1992
	Megeve Station De Praz-Sur-Arly	Biofor C	3	306	7.2	1999
	Megeve Station De Praz-Sur-Arly	Biofor CN	6	306	3.3	
	Metabief	Biofor C	4	113	0.6 (1.6)	1985
	Nesle	Biofor C	3	172	1.4	1999
	Orsan-Amylum	Biofor C	3	176	1.4	1997
	Perigueux	Biofor CN	6	560	4.1 (9.5)	1992
	Petit Couronne	Biofor C	8	264	3.8	1991
	PWA Le Theil	Biofor C	4	151	1.3	1993
	Saint Palais	Biofor C	4	264	2.5 (5)	1984
	Saint Palais Extension	Biofor C	5	264	5.0 (7.0)	1990
	Sibille Dalle Dalle Et Lecomte	Biofor C	6	339	3.8	1995
	Toulouse	Biofor C	6	441	8.5 (10.6)	1989
	Valbonne-Sophia Antipolis Station Des Bouillides	Biofor C	4	264	1.4 (3.7)	1996

Biofor™ Installation List

	Installation	Process	Number of Filters	Filter Area (Ft^2/Cell)	Average (Peak) Flow (MGD)	Construction Start-Up
Germany	Ahlen	Biofor DN	8	506	3.8 (13)	1998
		Biofor N	8	560		
	Ahrensburg	Biofor DN	5	301	2.3	1996
		Biofor N	5	204		
	Bielefeld Bielfeld-Heepen	Biofor N	9	786	11.9 (22.2)	1992
	Bielefld Obere Lutter	Biofor N	10	441	8.7 (25.3)	1992
	Bissendorf	Biodrof CN	6	226	3.2 (4.8)	1992
	Brewery Veltins Grevenstein	Biodrof C	4	130	0.8 (1.3)	1987
	Buchmann Paper Mill	Biofor C	3	194	1.6 (3.0)	1995
	Cloppenburg	Biofor CN	16	441	3.8 (10.8)	1990
		Biofor N	7	441		
	Cologne Cologne-Porz Wahn	Biofor N	8	441	5.4 (18.3)	1991
	Cologne Cologne-Rodenkirchen	Biofor N	6	441	4.3 (13.4)	1993
	Cologne Cologne-Stammheim	Biofor N	48	786	82.4 (210.1)	1992
	Cologne Colon-Langel	Biofor N	7	441	5.3 (15.9)	1992
	Drewsen Paper Mill	Biofor C	4	252	1.5 (2.4)	1993
		Biofor CN	5	252		
	Elsdorf	Biofor N	4	205	3.8	1989
	Erftverband Elsdorf	Biofor N	4	205	1.1 (3.8)	1990
	Frankfurt	Biofor DN	7	785	79.3 (155.4)	1998
	Guetersloh Guetersloh-Putzhagen	Biofor N	9	441	22	1992
	Gutersloh	Biofor N	9	441	11.9 (22.2)	1992
	Haindl Paper Mill Walsum	Biofor C	5	237	5.0 (5.1)	1990
	Julius Glatz Mill Neidenfels	Biofor C	6	345	5.4 (7.6)	1989
	Kammerer Paper Mill	Biofor C	5	226	3.8 (4.8)	1996
	Koln	Biofor N	7	441	15.9	1992
	Konigstein Paper Mill	Biofor C	3	118	0.8 (0.95)	1996
		Biofor N	3	118		
	Lage-Lippe	Biofor N	7	441	4.1 (14)	1992
	Mannheim	Biodrof C	32	937	44.9 (150.3)	1987
	MD Paper Mill	Biofor C	7	441	4.4 (5.7)	1994
		Biofor C	7	441		
	Paper Mill Delligsen	Biodrof C	3	75	0.7 (0.8)	1985
	Paper Mill Mochenwangen	Biofor C	4	189	2.4 (2.9)	1990
	PWA Dekor Paper Mill	Biofor C	5	226	3.7 (4.8)	1990
	PWA-Stockstadt Paper Mill	Biofor C	5	441	5.8 (7.3)	1993
		Biofor N	5	441		
	Rostock	Biofor N	12	785	17.2 (44.4)	1996
		Biofor DN	12	785		
	Schoeller-Hoesch Paper Mill	Biofor C	4	441	7.6 (9.5)	1995
	Supplingenburg	Biofor N	2	237	1.2 (1.7)	1991
	Temming Paper Mill Gluckstadt	Biofor C	4	189	1.3 (1.6)	1991
	Thalheim	Biofor DN	3	258	0.7 (2.9)	1997
		Biofor N	3	258		
	Vlotho	Biofor DN	4	247	3.8	1998
		Biofor N	4	301		
	WNC-Nitrochemie	Biofor CN	1	226	0.7 (1.0)	1994
		Biofor DN	2	226		
	Wuerselen	Biofor DN	3	152	1.6 (7.3)	1996
Spain	Cadix - Santa Maria	Biofor C	7	753	71.3 (146.5)	1994
		Biofor N	4	753		
	Canary Islands	Biofor C	2	258	9.5	1994
	Guimar	Biofor C	2	258	1	1993
	Ibiza	Biofor DN	4	560	11.4	1999
	Puerto de Santa Maria	Biofor C	11	753	7.1	1993
	San Antonio/San Jose Ibiza	Biofor C	6	441	4.9	1992
Canada	Canmore, Alberta	Biofor C	4	431	2.5 (7.3)	1995
		Biofor N	4	431		
	Chateauguany	Biofor C	12	527	7.2 (24.8)	1991
	Quebec	Biodrof	52	893	41.4 (109)	1992
	Royal Polymers, LTD	Biofor C	3	29	2.2	2002
	Thunder Bay Ontario	Biofor C	8	1151	70	2005*
		Biofor N	6	1151		
	Windsor, Ontario	Biofor CN	16	1506	121	2006*

Biofor™ Installation List

Country	Installation	Process	Number of Filters	Filter Area (Ft^2/Cell)	Average (Peak) Flow (MGD)	Construction Start-Up
United Kingdom	Ayshire Meadowhead	Biofor C	8	931	12	2000
	Ashford	Biofor N	4	440	6.6 (9.5)	2004
		Biofor DN	4	110		
	Birkenhead	Biofor C	7	1518	225	1997
	Bromborough	Biofor C	6	1518	190	1997
	Crewekerne East	Biofor N	4	110	1.4 (1.6)	2001
	Flag Fen	Biofor N	6	1137	25.1 (37.4)	1997
	Langford Recycling	Biofor N	3	786	9.25 (12)	2001
		Biofor DN	3	302.5		
	Menagwins, St. Austell	Biofor N	4	209	1.7 (4.3)	1996
	Plymouth	Biofor C	4	786	16	1998
	Poole	Biofor C	8	786	7.4 (17)	1995
		Biofor N	18	786		
	Sandown Secondary/Isle of Wight	Biofor C	6	1378	27.3 (33.7)	2001
	Torbay	Biofor C	8	786	29.3 (35.9)	1999
Italy	Comodepur-Come	Biofor C	2	301	1.9	1998
		Biofor N	2	301		
		Biofor DN	2	114		
	Peschiera - Borromeo	Pre-DN	10		31.7 (63.4)	2002
		Biofor N	10			
	Pocari Paper Mill	Biofor C	10	344	10.8	1992
	Pulsano	Biofor C	6	244	4.6 (13.6)	2000
		Biofor N	6	306		
		Biofor DN	3	184		
	Riva del Garda	Biofor C	6	301	4.1	1997
	Sesto San Giovanni	Biofor C	4	477	6.5 (21.5)	1998
		Biofor N	4	562		
		Biofor DN	3	477		
	Sinoaco Paper Mill	Biofor C	4	103	1.6	1989
	Subiaco - Roma	Biofor C	4	103	1.6	1988
Switzerland	Daihan Swiss	Biofor C	3	301	1.3	1997
	Fribourg	Biofor N	8	603	33.1	1996
	Genf	Biofor	2	231	1.6	1988
	Knonau	Biofor N	3	129	3.7	1999
	Nyon	Biofor C	6	441	7.6	1993
	Perroy	Biofor C	2	188	1.9	1989
		Biofor N	1	188		
	Valley of Bagnes	Biofor C	4	352	5.4	1992
Other Countries	Australia - Sydney, Cronulla	Biofor C	2	135	0.6 (1.6)	1993
		Biofor N	2	135		
	Austria - Leather Factory Vogl	Biofor C	1	65	0.1	1992
	Austria - Sca Laakirchen Ag	Biofor C	8	226	7.3 (8.4)	1995
	Belgium - Cockerill Sambre	Biofor C	2	65	0.1 (0.4)	1994
	Belgium - Fabelta	Biofor C	7	245	1	1995
	Belgium - Morlanwelz	Biofor DN	2	306	3.4	1995
	China - Dalian	Biofor CN	12	786	31.7 (41.2)	1999
		Biofor N	12	786		
	Finland - Oulu	Biofor C	6	409	14	1998
	Hungary - Budapest	Biofor N	10	863	31.7 (50.7)	1996
		Biofor DN	6	631		
	Netherlands - Den Haag	Biofor	2	231	0.6	1992
	Norway - Oslo	Biofor CN	24	952	94 (129.3)	1996
		Biofor DN	24	694		
	Portugal - Loule-Algarue	Biofor N	3	245	3.2	1998
		Biofor DN	3	245		
	Portugal - Peniche	Biofor CN	4	241	2.2 (5.2)	2000
	Portugal - Sesimbra	Biofor C	4	245	1.6	1999
		Biofor N	2	245		
	Russia - Novorossyisk	Biofor DN	4	(steel)	1.3	2000
	Venezuela - Cardon	Biofor N	6	452	10.1 (25)	1988

* Denotes facilities not yet in operation

Biofor C = Carbonaceous removal

Biofor CN = Carbonaceous removal and Nitrification

Biofor N = Nitrification

Biofor DN = Denitrification

Krüger

BIOSTYR REFERENCE LIST

FULL-SCALE IMPLEMENTATION

No.	Year Started	Location	Average Design Flow (mgd)	Number of Cells	Area of Each Cell (ft^2)	Type of Treatment [1]						Effluent Requirement (mg/l) and/or Percent Removal					
						Carbon Removal	Secondary Nitrification	Tertiary Nitrification	Classical Nitrification-Denitrification	Simultaneous Nitrification-Denitrification	Denitrification	TN	TKN	NH_3-N	BOD	TSS	TP
1	1990	St. Jean d'Illac, France	0.55	5	172				X			20	5				
2	1992	Cergy Pontoise, France	10.6	16	678			X					10				
3	1992	Nyborg, Denmark	3.4	8	678				X			8					1.5
4	1993	Evreux, France	5.25	4	678			X					10				
5	1994	Melun, France	6.3	12	904				X			20					
6	1994	Frederikshavn, Denmark	2.62	5 + 1	678				X		X	8					1.5
7	1994	Hobro, Denmark	2.4	6	300				X			8					0.4
8	1995	Assens, Denmark	0.55	3	150		X						5				
9	1995	Toulouse Blagnac, France	2.62	6	678				X			15					2
10	1995	Lyon St. Fons, France	22.5	5	1195		X							2			
11	1995	Mariager, Denmark	0.75	4 + 2	150				X		X	8					
12	1995	Rome, Italy	50	16	1195		X							4			

No.	Year Started	Location	Average Design Flow (mgd)	Number of Cells	Area of Each Cell (ft^2)	Type of Treatment [1]						Effluent Requirement (mg/l) and/or Percent Removal					
						Carbon Removal	Secondary Nitrification	Tertiary Nitrification	Classical Nitrification-Denitrification	Simultaneous Nitrification-Denitrification	Denitrification	TN	TKN	NH_3-N	BOD	TSS	TP
13	1995	Rambouillet, France	28	8	300				X			15					2
14	1997	Alanya, Turkey	7.4	6	678		X							4			2
15	1997	Herford, Germany	8.7	16	1195				X			15					0.8
16	1997	Neuchatel, Switzerland	3.2	6	452	X											0.3
17	1998	Altenrhein, Switzerland	3.6	8	678				X			20					
18	1998	Araches, France	0.7	4	300		X						10				
19	1998	Biarritz Anglette Bayonne, France	2.9	6	452		X						10				
20	1998	Colombes, Paris, France	63	30	1195			X				10					1
21	1998	Davyhulme, Manchester, Great Britain	98.25	36	1195			X						5			
22	1998	Frielas, Portugal	18.5	8	1195	X		X							10	15	
23	1998	Kauhajoki, Finland	2.62	4	140			X						8			
24	1998	Nyborg Extension, Denmark	4.25	7 + 3	678						X	7					
25	1998	Hobro Extension, Denmark	2.4	6 + 2	300						X	8					0.4
26	1999	Barcelonnette, France	2.4	8	300		X							2			
27	1999	Painscarolo, Switzerland	1.8	5	300			X						1			

No.	Year Started	Location	Average Design Flow (mgd)	Number of Cells	Area of Each Cell (ft^2)	Type of Treatment [1]						Effluent Requirement (mg/l) and/or Percent Removal					
						Carbon Removal	Secondary Nitrification	Tertiary Nitrification	Classical Nitrification-Denitrification	Simultaneous Nitrification-Denitrification	Denitrification	TN	TKN	NH_3-N	BOD	TSS	TP
28	1999	Pargas, Finland	3.2	6 + 4	140				X		X	7					
29	2000	Colombiers, Switzerland	4.7	5	678		X							No Limit. Achieve >2.0	20	10	0.8
30	2000	Toulouse Ginestous, France	44	10	1195			X						5			
31	2000	Lagny, France	18.34	8 + 7	904				X		X	10					
32	2000	Fazakerley, Great Britain	11	7	1215			X						2	10		
33	2001	Geneva, Switzerland	74 Dry 139 Pk.	20	1862		X							2	>90% Removal	15	2
34	2001	Freeport, IL, USA	7.1 (18.2 peak)	7	470			X						1.1	10	10	
35	2001	Oldham, Great Britain	13.2	8	1215			X						3	10		
36	2001	Countess Wear, Great Britain	13.9	5	1215			X						10			
37	2001	Le Mans, France	11.9 (29.1 Pk.Day)	18 + 2	1215				X		X	15 (10 Ann. Avg.)	8	5	25	15	2 (1.0 Ann. Avg.
38	2002	Northfield, MN, USA	3.4 (11.4 peak)	10	470		X							8	16	20	1
39	2002	L'Argentiere, France	0.4	3	130					X		15					
40	2002	Chapel-en-le-Frith, Great Britain	1.6	3	150												
41	2002	Shepton Mallet, Great Britain	1.9	5	130			X						4			
42	2003	Lyss, Switzerland	7	7	904				X			55% removal		2			0.8

No.	Year Started	Location	Average Design Flow (mgd)	Number of Cells	Area of Each Cell (ft^2)	Type of Treatment [1]						Effluent Requirement (mg/l) and/or Percent Removal					
						Carbon Removal	Secondary Nitrification	Tertiary Nitrification	Classical Nitrification-Denitrification	Simultaneous Nitrification-Denitrification	Denitrification	TN	TKN	NH_3-N	BOD	TSS	TP
43	2003	Boulogne, France	19.2	9+4	1215 & 495					X	X	10					
44	2003	Saint Peter, MN, USA	2.3 (10 peak)	6	470		X							5	15	30	1
45	2003	Helsinki, Finland	68.8	10	1582						X	6 or 70% removal					
46	2003	Luneville, France	2.4	4 + 6	678 + 430					X	X						
47	2003	Sarregumines, France	4.14	7	904				X			15					
48	2003	Res Nabi Younes, Lebanon	1.4	5	300												
49	2003	Douarnenez, France	1.57	5 + 1	678 + 300					X	X	15					
50	2003	Uusikaupunki, Finland	1.9	6 + 4	300 + 140		X				X	70% removal					
51	2003	Kajani, Finland	2.62	8	300												
52	2003	Montpelier, France	29.61	8	1862												
53	2003	Farum, Denmark	1.44	4	140						X	5					
54	2003	La Barossa, Spain	1.31	4	452					X		15					
55	2004	Syracuse, NY, USA	84 (126.3 peak)	18	1585			X						2	10	20	
56	2004	Bern, Switzerland	29.85	16	1250				X			15 or 55% removal					
57	2004	Denver, CO - USA	7.4 Avg. (30 Peak)	7	705			X						1 (30 day) 2 (daily)			

No.	Year Started	Location	Average Design Flow (mgd)	Number of Cells	Area of Each Cell (ft^2)	Type of Treatment [1]						Effluent Requirement (mg/l) and/or Percent Removal					
						Carbon Removal	Secondary Nitrification	Tertiary Nitrification	Classical Nitrification-Denitrification	Simultaneous Nitrification-Denitrification	Denitrification	TN	TKN	NH_3-N	BOD	TSS	TP
58	2004	Songdo, Korea	2.62	4 + 3	678 + 140					X	X	25			15	12	
59	2004	Formigueres, Frances	0.75	4 + 1	452 + 150					X	X	15					
60	2004	Savio, Italy	1.57	8	452				X			15			25	35	
61	2004	Cluses, France	8.38	6	678		X							24	25	35	
62	2005	Acheres, France	445	84	1851			X						4			
63	2005	Tahoe Truckee, CA, USA	16.77	8 + 4	940 + 705			X			X	3					
64	2006	Cheshire, CT, USA	3.5	5	150						X	1.5 (NOx)					
65	2006	Mystic Lake, MN, USA Phase I	0.64	4	304		X							5	25	30	1
66	2007	McCook, NE, USA	1.75	4	150			X						0.9	10	12	

Note 1
Carbon = carbonaceous removal
Secondary Nitrification = secondary nitrification and carbon removal
Classical Nitrification/Denitrification = nitrification and denitrification in the same unit with two air grids
Simultaneous Nitrification/Denitrification = nitrification and denitrification in the same unit with internal recirculation and single air grid
Tertiary Nitrification = tertiary nitrification
Denitrification = post denitrification with external carbon source

Appendix C
Partial List of TSFF Installations

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MOVING BED BIOFILM REACTOR MUNICIPAL INSTALLATION LIST

PLANT & LOCATION		SIZE P.E.	REACTOR VOLUME (m3)	(ft3)	OBJECTIVES
Steinsholt Norway	1990	625	50	1,766	N-removal, Pre-denitrification
Eidsfoss Norway	1992	500	52	1,836	BOD-removal, 1,000 P.E. max.
Harran Norway	1992	600	5	177	BOD-removal
Bekkelaget Oslo	1992	15,000	595	21,009	N-removal, Post-denitrification
Tana-Bru Norway	1993	1,750	99	3,496	Aerobic reactor, BOD-removal
Karasjok Norway	1993	4,000	87	3,072	Aerobic reactor, BOD-removal
Risby Norway	1993	70	5	177	BOD removal
Lillehammer Norway	1994	70,000	3,840	135,590	N-removal, Pre/Post-denitrification
Vrigstad Sweden	1994	2,300	114	4,025	Replacement of activated sludge
Farstorp Sweden	1994	200	22	777	Replacement of activated sludge
Saleboda Sweden	1994	700	22	777	Aerobic reactor, BOD-removal
Sanderstolen Norway	1994	350	19	671	BOD-removal
Siljan Norway	1995	2,200	110	3,884	BOD-removal
Dejtar Hungary	1995	2,000	206	7,274	N-removal, Pre-denitrification
Kishartyan	1995	1,500	139	4,908	N-removal,

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MOVING BED BIOFILM REACTOR MUNICIPAL INSTALLATION LIST

PLANT & LOCATION		SIZE P.E.	REACTOR VOLUME (m3)	(ft3)	OBJECTIVES
Hungary					Pre-denitrification
Mediå, Grong Norway	1995	1,700	100	3,531	BOD-removal
Bury St. Edmunds UK	1995	17,000	500	17,655	Nitrification
Doddington UK	1995	3,600	300	10,593	BOD-removal
Spiken Sweden	1995	900	35	1,236	BOD-removal
Munkedal Sweden	1995	7,000	230	8,121	BOD-removal
Homestrand Norway	1996	15,000			COD and P removal
Deje Sweden	1996	4,200	100	3,531	BOD-removal & DAF
Byrkjelo Norway	1996	2,850	33	1,165	BOD-removal
Anwick STW UK	1996	N/A	1,800	63,558	Replacement of Fixed Media Process
Dunwick UK	1996	250	20	706	BOD and nitrification
Norde Follo Norway	1997	40,000	3,700	130,647	N-removal, Pre/Post-denitrification
Skara, Odda Norway	1997	500	12	424	BOD and phosphorus Removal
Røldal, Odda Norway	1997	700	14	494	BOD and phosphorus Removal
Røra, Inderøy	1997	7,500	179	6,320	BOD-removal

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MOVING BED BIOFILM REACTOR MUNICIPAL INSTALLATION LIST

PLANT & LOCATION		SIZE P.E.	REACTOR VOLUME (m3)	(ft3)	OBJECTIVES
Norway					
Plaza Indonesia R. Indonesia	1997	1,800	170	6,003	BOD-removal
Bjuv Sweden	1997	16,000	171	6,038	Nitrogen removal post-denitrification
Derby Pride UK	1997	N/A	545	19,244	BOD-removal
Spåtind Norway	1997	250	8	282	BOD-removal
Western Plant Wellington, New Zealand	1997	11,000	350	12,359	BOD-removal
Klagshamn Sweden	1997	90,000	171	6,038	N-removal, Post-denitrification
Laufäcker, Baden Switzerland	1997	10,000	835	29,484	N-removal, Pre-denitrification
Nettleham UK	1997	4,800	316	11,158	BOD-removal
Öckerö Sweden	1997	14,000	439	15,501	N-removal, Post-denitrification
Tuddenham UK	1997	1,000	37	1,306	Nitrification
Skepshult Sweden	1997	600	30	1,059	BOD-removal
Hallabro Sweden	1997	300	14	494	BOD-removal
Linneryd Sweden	1997	600	80	2,825	BOD-removal
Moa Point	1998	200,000	2,760	97,456	BOD-removal

MOVING BED BIOFILM REACTOR MUNICIPAL INSTALLATION LIST

PLANT & LOCATION		SIZE P.E.	REACTOR VOLUME (m3)	(ft3)	OBJECTIVES
Wellington, New Zealand					
Gardermoen Norway	1998	50,000	5,790	204,445	N-removal, Pre/Post-denitrification
Nyköping Sweden	1998	70,000	3,660	129,235	N-removal, 15 mg/L Pre-denitrification
Braintree STW UK	1998	28,000	2,360	83,332	Upgrade with K2 N-removal
Corby STW UK	1998	240,000	4,000	141,240	Increased capacity for BOD-removal
Great Dunmow STW UK	1998	8,000	650	22,952	Upgrade with BOD & ammonia removal
Velkua Kunta Finland	1998	100	12	424	BOD-removal
Bury St. Edmunds II UK	1998	17,000	1,000	35,310	Nitrification
Pyewipe UK	1998	314,000	3,960	139,828	BOD-removal
Naprava, Domzale Slovenia	1998	N/A	500	17,655	Test Plant, N-removal
Penig Germany	1998	50	6	212	Upgrade BOD-removal
Burgsvik Sweden	1998	2,000	45	1,589	BOD-removal
Strängnäs Sweden	1998	25,000	1,000	35,310	Nitrification & Post-denitrification
Näsum Sweden	1998	500	25	883	BOD-removal

AnoxKaldnes

MOVING BED BIOFILM REACTOR MUNICIPAL INSTALLATION LIST

PLANT & LOCATION		SIZE P.E.	REACTOR VOLUME (m3)	(ft3)	OBJECTIVES
Ljusdal Sweden	1998	12,500	56	1,977	BOD-removal
Sjölunda Sweden	1998	375,000		-	Denitrification
Lisifhaus Wildhaus I Switzerland	1998	500	18	636	Compact Plant Replace Disc Filter
Shoreham UK	1999	N/A	115	4,061	Treatment of Reject Water
Frya Norway	1999	9,000	176	6,215	BOD-removal
Tretten Norway	1999	4,300	108	3,813	BOD-removal
Fislisbach Switzerland	1999	9,900	1,063	37,535	Nitrogen Removal
Sernftal, Engi Switzerland	1999	3,000	202	7,133	Compact Plant Replace Disc Filter
Margretelund Sweden	1999	40,000	2,750	97,103	Nitrification & Pre/Post-denitrification
Caboolture Australia	1999	40,000	200	7,062	N-removal, Post-denitrification
Colchester UK	1999	110,000	1,378	48,657	Roughing reactor ahead of AS
Svarstad Norway	1999	2,000	90	3,178	BOD removal Replace AS plant
Vindfjelltunet Norway	1999	200	7	247	BOD removal Tourist plant
BAS/SIAD Bergamo, Italy	1999	45,000	1,340	47,315	Nitrification after AS

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MOVING BED BIOFILM REACTOR MUNICIPAL INSTALLATION LIST

PLANT & LOCATION		SIZE P.E.	REACTOR VOLUME (m3)	(ft3)	OBJECTIVES
Fyresdal Norway	2000	2,800	46	1,624	BOD removal, Replace AS plant
Mooya Norway	2000	5,000	81	2,860	BOD removal
Bruch Germany	2000	9,800	830	29,307	Plant Upgrade, BOD Removal - Nitrification
Knivsta Sweden	2000	15,000	560	19,774	Nitrification
Tafalla & Olite Spain	2000	34,300	850	30,013	Upgrade roughing reactor & Partial nitrification
Colchester UK	2000	164,000	1,400	49,434	BOD roughing
Chiba Prefecture Japan	2000	3,500	247	8,721	W.W. Treatment
Vassenden Norway	2000	1,500	52	1,836	BOD removal
Mattarello Italy	2000	6,000	271	9,570	BOD removal
Cala Gonone Italy	2000	15,000	256	9,040	BOD removal
Bekkelaget Norway	2001	350,000	1,325	46,786	Activated Sludge w/ Kaldnes - Post-denit
Visby Sweden	2001	50,000	550	19,420	Post-Denitrification
Vanersborg Sweden	2001	31,000	250	8,827	Post-Denitrification
Dervio Italy	2001	14,000	519	18,326	Nitrification, Pre & Post Denitrification

MOVING BED BIOFILM REACTOR MUNICIPAL INSTALLATION LIST

PLANT & LOCATION		SIZE P.E.	REACTOR VOLUME (m3)	(ft3)	OBJECTIVES
Tauro Gran Canary	2001	2,000	48	1,695	BOD removal
Beitostolen Norway	2001	9,700	120	4,237	BOD removal
Vihti Finland	2001	1,600	150	5,300	Replacement of AS to Achieve nitrification
RA-2 Norway	2002	160,000	19,000	670,890	N-removal, Pre & Post Denitrification
Highland Creek WWTP Toronto, Canada	2002	45,000	1,190	42,100	Nitrification
Broomfield WWTP Colorado, USA	2002	80,000	4,867	171,864	Hybrid Nitrification
Hveragerdi Iceland	2002	4,500	60	2,120	BOD removal
Byrkjelo Norway	2002	8,000	160	5,650	BOD removal
Delphi (ETE Jambeiro) Brazil	2002	700	43	1,520	BOD removal
South Adams County Colorado, USA	2003	50,000	4,640	164,000	Nitrogen Removal, Pre-Denitrification & Nitrification
Fyresdal Norway	2003	2,400	82	2,895	BOD removal
Bjorkelangen Norway	2003	8,000	151	5,330	BOD removal
Skreia Norway	2003	9,300	162	5,720	BOD removal
Merrimac WWTP Wisconsin, USA	2003	460	390	13,824	Pre-denitrification

AnoxKaldnes

MOVING BED BIOFILM REACTOR MUNICIPAL INSTALLATION LIST

PLANT & LOCATION		SIZE P.E.	REACTOR VOLUME (m3)	(ft3)	OBJECTIVES
Johnstown WWTP Colorado, USA	2003	4,000	1,530	54,000	Nitrification
Poipu WWTP Poipu, Hawaii USA	2003	1,200	374	13,200	BOD Removal
Cheyenne Crow Creek WWTP	2004	31,000	7,825	276,300	Nitrogen removal Pre-denitrification Nitrification
Cheyenne Dry Creek WWTP	2004	22,000	5,580	197,000	Hybrid Nitrification

Appendix D
Partial List of NTF Installations

ACCU-PAC® SURFACE MEDIA

PARTIAL LIST OF WASTEWATER TREATMENT INSTALLATIONS NITRIFICATION BIOFILTERS

PROJECT/CUSTOMER	TYPE OF APPLICATION	QUANTITY	DATE OF INSTALLATION/SUPPLY
City of Olathe, KS	BOD, Nitrification	51,000 cu.ft.	10/87
Village of Shiloh, OH	BOD, Nitrification	4,350 cu. ft	6/91
City of Lafayette, IN	Nitrification	15,000 cu.ft.	11/95
Central Valley, UT	Nitrification	967,000 cu.ft	6/96
Sturgis, MI	Nitrification	76,800 cu.ft.	5/97
Littleton-Englewood	Nitrification	208,000 cu.ft.	6/98
Austin, MN	Nitrification	765,000 cu.ft.	9/98
Napoleon, OH	Nitrification	150,000 cu.ft.	9/98
Washington. PA	Nitrification	65,900 cu.ft.	9/98
Richmond, KY	Nitrification	46,800cu.ft.	7/99
Alfred, NY	Nitrification Filter	49,000 cu. ft.	5/00
Englewood, CO	Nitrification	17,568 cu.ft	11/00
Dayton, OH	Nitrification	157,000 cu.ft	12/00
Dayton, OH	Nitrification	381,000 cu.ft	2/01

BRENTWOOD INDUSTRIES
610 Morgantown Road (19611)
PO Box 605
Reading, PA 19603 USA
phone: (610)-374-5109
fax: (610)-376-6022

City of Benicia-Water Reuse Project Draft Technical Memorandum No. 2 - Evaluation of Alternative Disinfection Processes

To: Chris Tomasik

CC: PURE Members

DATE: November 4, 2004

Executive Summary

The purpose of this TM is to determine the most cost-effective disinfection system for the Benicia Water Reuse Project. The TM presents the results of the evaluation of selected technologies for disinfection of recycled water produced by the micro-filtration and reverse osmosis process. Conceptual designs and evaluations of alternative disinfection systems were developed to comply with the various regulatory requirements.

Alternative disinfection systems evaluated for the Benicia Reuse Project include chlorination using sodium hypochlorite and ultraviolet light disinfection. Recycled water from the proposed Water Reuse Treatment System must meet disinfection requirements for tertiary recycled water, proposed for use as cooling water supply, as contained in Title 22, Division 4, Chapter 3 of the California Code of Regulations (Title 22).

For either chlorination or UV disinfection, Title 22 requires that the median concentration of total coliform bacteria measured in the disinfected effluent shall not exceed 2.2 per 100 milliliters over the prior seven-day test period, not exceed 23 per 100 milliliters in more that one sample in any 30-day period, and never exceed 240 total coliform bacteria per 100 milliliters. Title 22 requires that a chlorine disinfection process must provide a CT (the product of chlorine residual and modal contact time) value of not less than 450 milligram-minutes per liter at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow. Generally, this results in a design hydraulic residence time of 120 min. Title 22 requires demonstration that alternative disinfection systems, such as UV, when combined with the filtration process, inactivate and/or remove 99.999 percent (5 log inactivation or removal) of the plaque-forming units of F-specific bacteriophage MS2, or polio virus in the wastewater. In addition, the micro-filtration process must meet the Title 22 turbidity performance requirements for micro-filtration which require that the filtered water does not exceed 0.2 NTU more than 5% of the time within a 24-hour period, and 0.5 NTU at any time.

The State Department of Health Services (DOHS) is responsible for approving UV disinfection systems and bases its approval of UV systems on the *Utraviolet Disinfection – Guidelines for Drinking Water and Water Reuse.* All UV disinfection systems proposed for water reuse in California must be validated under the 2003 NWRI/AWWARF Guidelines, which contain extensive design criteria that are discussed in the body of this TM.

Types of UV systems are defined by the type of mercury vapor lamp used to produce the light and the configuration of the lamps used to apply the light to the process flow. The three types of UV lamps, and hence types of UV disinfection systems, are the following:

- Low Pressure, Low Intensity Lamps
- Medium Pressure, High Intensity Lamps
- Low Pressure, High Intensity Lamps

In recycled water applications these systems are generally all installed in open channels. All of these systems use ultraviolet light to destroy the ability of pathogenic organisms to reproduce, thus eliminating their potential for infection. The design parameter for UV disinfection is dose, which is the product of UV intensity and exposure time. For recycled water that has undergone micro filtration, the required design dose is 80 micro-Joules per centimeter squared (mJ/cm^2).

The three types of UV systems were reviewed for applicability to the Benicia Water Reuse Project. Low Pressure, High Intensity (LPHI) was selected owing to energy efficiency, number of lamps required and applicability to the size of this project.

Four (4) overall disinfection system alternatives were developed and evaluated, as outlined below. There are two differently sized systems for each disinfection process alternative because the possibility exists that the MF process could be located at the City's WWTP and the RO process could be located at the Valero Refinery. If the RO system is located at the Refinery, then the disinfection system would need to have a capacity of 2.4 mgd to allow for the reject flow of concentrate in order to generate 2.0 mgd of recycled water.

- 2.0 mgd Chlorination at the site of the City's WWTP after MF/RO
- 2.4 mgd Chlorination at the site of the City's WWTP after only MF
- 2.0 mgd LPHI UV at the site of the City's WWTP after MF/RO
- 2.4 mgd LPHI UV at the site of the City's WWTP after only MF

For each of the four, overall disinfection alternatives, conceptual designs were prepared and construction and O&M cost estimates were developed. For each of the UV system alternatives some chlorination is also required to prevent slime growths in the transmission pipeline. Table ES-1 below summarizes these estimated costs.

Table ES-1
Summary of Present Worth Analysis for Alternative Disinfection Systems

	Alternative Disinfection Systems			
	2.0 mgd CL	**2.0 mgd UV**	**2.4 mgd CL**	**2.4 mgd UV**
	$1,000s(1)	**$1,000s(1)**	**$1,000s(1)**	**$1,000s(1)**
Estimated Construction Costs(2)	$980	$1,070	$1,190	$1,070
25% Allowance for Engineering, Admin and Legal Costs	$250	$270	$300	$270
Total Estimated Capital Costs	$1,230	$1,340	$1,490	$1,340
Estimated Annual O&M Costs(3)	$77	$85	$110	$88
PW of O&M Costs(4)	$880	$970	$1,260	$1,010
Total Estimated Present Worth(5)	**$2,110**	**$2,310**	**$2,750**	**$2,350**

(1) All Values have been rounded to the closest $10,000
(2) See Tables 5-2 and 6-2 for chlorine and UV estimated construction costs, respectively
(3) See Tables 5-4 and 6-4 for chlorine and UV estimated O&M costs, respectively
(4) O&M Cost times Present Worth Factor for 20 years at 6% interest. PWF=11.47.
(5) Equals the sum of Estimated Capital Cost and PW of Estimated O&M Cost

Table ES-2
Summary of Qualitative Evaluation of Chlorination and UV Disinfection

Qualitative Factors	**Chlorination**	**UV Disinfection**
Impact on Existing Facilities	High	Slight
Ease of Operation	Moderate	Moderate
Flexibility for Changing Requirements	Low	Good
Ease of Implementation	Moderate	Good
Future Expandability	Low	Good
Equipment Reliability	Good	Good
Process Reliability	Variable	Good
Proven Technology	High	Moderate
Process Complexity	Moderate	High
Impacts on Cooling Water Quality	Adverse	None
Safety	Adequate	Good
Public Acceptance	Adequate	Good

Conclusions and Recommendations

Based on the conceptual designs and economic analysis presented herein, the following conclusions are drawn:

- UV and chlorination appear to be nearly equally cost effective for the two sizes of systems evaluated, namely 2.0 mgd and 2.4 mgd, given the accuracy of the conceptual estimates, upon which they are based.
- Qualitative factors, in particular water quality impacts, site impacts and ease of process control, favor UV over chlorination

Based on the above conclusions, CDM recommends that the City select the low pressure, high intensity UV system as the preferred disinfection system for the Benicia – Valero Water Reuse Project.

1.0 Introduction and Purpose of the Technical Memorandum

A joint Water Reuse Project is being undertaken by the City of Benicia and the Valero Refinery to supply approximately 2 mgd of recycled water for cooling water make up at the Refinery.

TM 1, dated September 2004, evaluated alternative treatment processes to meet Valero's cooling water mineral requirements. The results of the evaluation were that the MF/RO process is the applicable water reuse treatment system that will meet Valero's water quality requirements. This Disinfection TM presents the results of the evaluation of selected technologies for disinfection of recycled water produced by the micro-filtration and reverse osmosis process. Conceptual designs and evaluations of alternative disinfection systems were developed to comply with the various regulatory requirements, which are discussed herein.

The purpose of this TM is to determine the most cost-effective disinfection system for the Benicia Water Reuse Project.

2.0 Applicable Regulatory Requirements

2.1 Title 22 Requirements

Alternative disinfection systems evaluated for the Benicia Reuse Project include chlorination and ultraviolet light disinfection. Recycled water from the proposed Water Reuse Treatment System must meet disinfection requirements for tertiary recycled water, proposed for use as cooling water supply, as contained in Title 22, Division 4, Chapter 3 of the California Code of Regulations (Title 22).

For either chlorination or UV disinfection, Title 22 requires that the median concentration of total coliform bacteria measured in the disinfected effluent shall not exceed 2.2 per 100 milliliters over the last 7 days of analyses, not exceed 23 per 100 milliliters in more than one sample in any 30 day period, and never exceed 240 total coliform bacteria per 100 milliliters.

2.1.1 Chlorination

Title 22 requires that a chlorine disinfection process must provide a CT (the product of chlorine residual and modal contact time[1]) value of not less than 450 milligram-minutes per liter at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow (2.0 mgd in this case). Generally, this results in a design hydraulic residence time of 120 min because short circuiting generally occurs which shortens actual hydraulic residence time.

The required CT value was discussed with Jeff Stone of DOHS for wastewater that receives MF and RO treatment. DOHS advised that there would be no credit for MF. There could be a one log virus reduction for RO; however, since it is proposed to split treat the RO with 15% MF filtrate, no reduction would be allowed.

2.1.2 UV Disinfection

Title 22 requires that UV equipment manufacturers conduct virus removal studies under the supervision of DOHS that demonstrate that their UV equipment, when combined with the filtration process, inactivate and/or remove 99.999 percent (5 log inactivation or removal) of the plaque-forming units of F-specific bacteriophage MS2, or polio virus in the wastewater.

2.1.3 Micro-Filtration

In addition, the micro-filtration process must meet the Title 22 turbidity performance requirements for micro-filtration which require that the filtered water does not exceed 0.2 NTU more than 5% of the time within a 24-hour period, and 0.5 NTU at any time.

2.2 DOHS Requirements for UV Disinfection

The State Department of Health Services (DOHS) is responsible for approving UV disinfection systems and bases its approval of UV systems on the *Utraviolet Disinfection – Guidelines for Drinking Water and Water Reuse,* prepared by the NWRI and the American Water Works Association Research Foundation (AWWARF), dated May 2003 (hereinafter referred to as "NWRI Guidelines"). All UV disinfection systems proposed for water reuse in California must be validated under the 2003 NWRI/AWWARF Guidelines

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1 "Modal contact time" means the amount of time elapsed between the time that a tracer, such as salt or dye, is injected into the influent at the entrance to a chamber and the time that the highest concentration of the tracer is observed in the effluent from the chamber.

The NWRI Guidelines contain design criteria as follows:

- Design for UV transmittance of 90% for RO and 65% for MF.
- Apply a lamp aging factor of 50%.
- Apply an additional fouling factor of 80%.
- Design on a bioassay-certified UV dose of 50 mJ/cm2 for RO and 80 for MF.
- Apply maximum allowable scale-up factor of 10 based on the UV manufacturers' pilot studies that led to their conditional acceptance by DOHS.

The NWRI/AWWARF Guidelines indicate that 5-log inactivation is achievable with a properly designed UV system so this option should not be a problem for UV systems.

CDM discussed the UV design criteria for UV design dosage and UV transmittance (UVT) with Rick Sakaji of DOHS. Because of the 15% split treatment of RO, the design must adhere to the requirements for MF. If, however, adequate operating UVT data are acquired and demonstrate that a design UVT greater than 65% could be used, then DHS would approve a system with a higher design UVT. For preliminary planning, CDM has used a design UVT value of 65% as required by the NWRI Guidelines. Based on experience with other UV projects, it is expected that full scale system may actually produce higher UVT values.

3.0 Description of Alternative Disinfection Systems

3.1 Bases for Development of Alternatives

Based on the estimated cooling water demand of 2 mgd and the regulatory requirements discussed in Paragraph 2.0 design criteria were developed for four disinfection alternatives, as follows:

- Chlorination at the site of the City's WWTP after MF/RO (disinfection system sized at 2 mgd)
- Chlorination at the site of the City's WWTP after only MF (disinfection system sized at 2.4 mgd)
- UV at the site of the City's WWTP after MF/RO (disinfection system sized at 2 mgd)
- UV at the site of the City's WWTP after only MF (disinfection system at 2.4 mgd)

There are two differently sized systems for each disinfection alternative because the possibility exists that the MF process could be located at the City's WWTP and the RO process could be located at the Valero Refinery. If that were the case, then it might be more appropriate to have the disinfection process on the City's site, so if there were a release of recycled water from the transport system, it would pose less public health concerns than if an event occurred with non-disinfected wastewater.

If the disinfection process follows the MF process then it must be sized for a design flow in the range of 2.30 to 2.43 mgd, to allow for reject rates in the range of 15% to 20% from the RO process. Figure 3.1 shows a preliminary flow balance diagram for the MF/RO process at two RO rejection rates. Hence, for this evaluation disinfection alternatives were sized for 2.0 mgd after RO and 2.4 mgd if disinfection occurs after MF.

Figures 3.2 and 3.3 present preliminary process block diagrams of the 2.0 mgd and 2.4 mgd disinfection alternatives, respectively, and Table 3-1 contains a summary of the regulatory design criteria for the four alternatives.

Table 3-1
Regulatory Design Criteria for Alternative Disinfection Systems

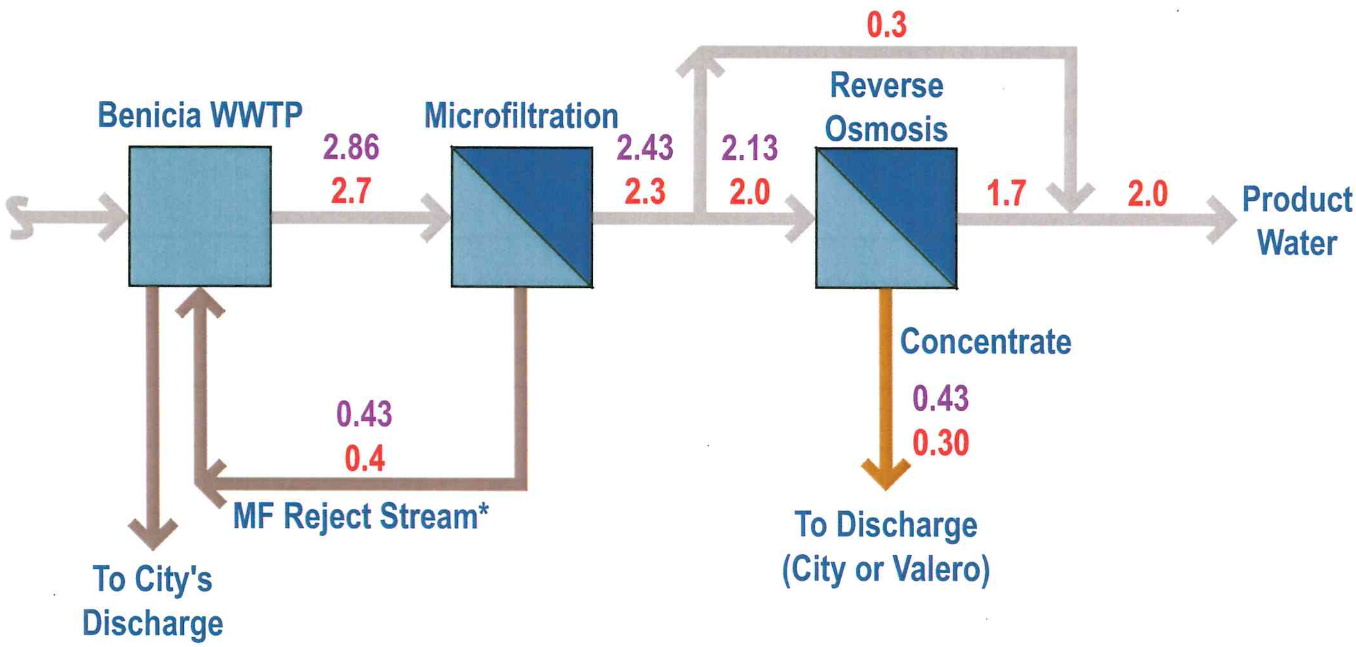
Design Parameter	*MF/RO + Cl2*	*MF + Cl2*	*MF/RO + UV*	*MF + UV*
Design Flow for the Disinfection Process, mgd	2	2.4	2	2.4
Bacteria Limitation, Total coliform MPN/100/mL	$\leq$ 2.2	$\leq$ 2.2	$\leq$ 2.2	$\leq$ 2.2
Virus Reduction, No. Logs	(1)	(1)	5	5
Turbidity (after MF), NTU	na(2)	na(2)	< 0.2	< 0.2
CT Value for Chlorination, mg-min/L	450	450	na(2)	na(2)
Dose for UV, mJ/cm^2	na(2)	na(2)	80	80
UV Transmittance, %	na(2)	na(2)	65	65

(1) DOHS considers designs composed of CT = 450 mg-min/L and HRT of 120 min to fulfill virus reduction requirements.

(2) na = not applicable.

3.2 Chlorination

Disinfection by chlorination is achieved by maintaining a minimum residual chlorine concentration in the process flow stream for a specific contact time. Facilities required for chlorination are chemical storage and handling facilities and a chlorine contact tank. Typically, chlorine contact tanks are cast-in-place concrete tanks and are relatively shallow and baffled to maximize contact time per unit of tank volume. Required chlorine contact tank volumes increase as flow and disinfection requirements increase.



All values are flow rates in mgd

*Assumed MF Reject Rate of 15%

(1) Assumed RO Reject Rate of 20%

(2) Assumed RO Reject Rate of 15%

Figure 3.1
Preliminary Flow Balance for MF/RO Water Reuse Treatment System

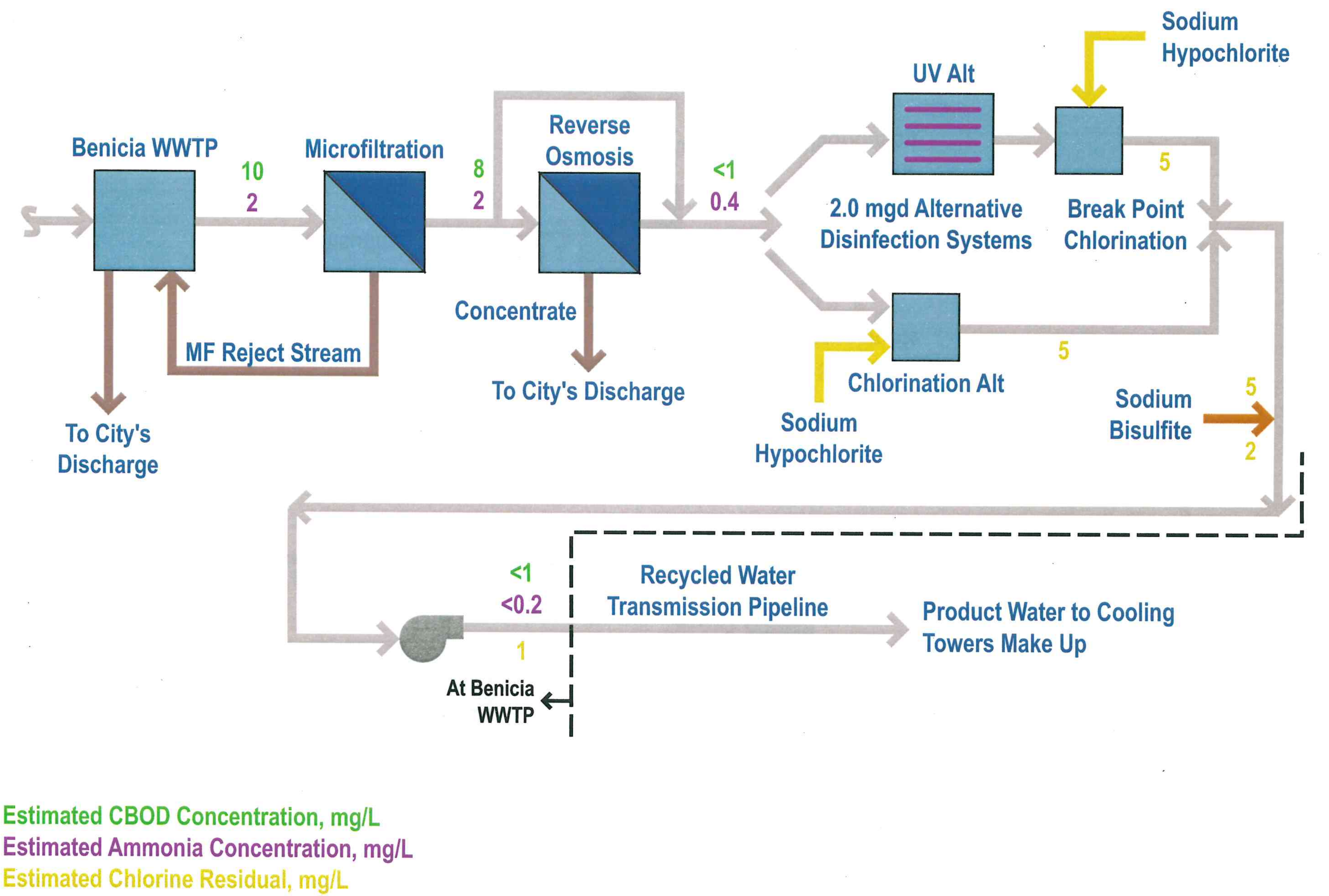


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Figure 3.2

Preliminary Process Block Diagram for 2.0 mgd Alternative Disinfection Systems

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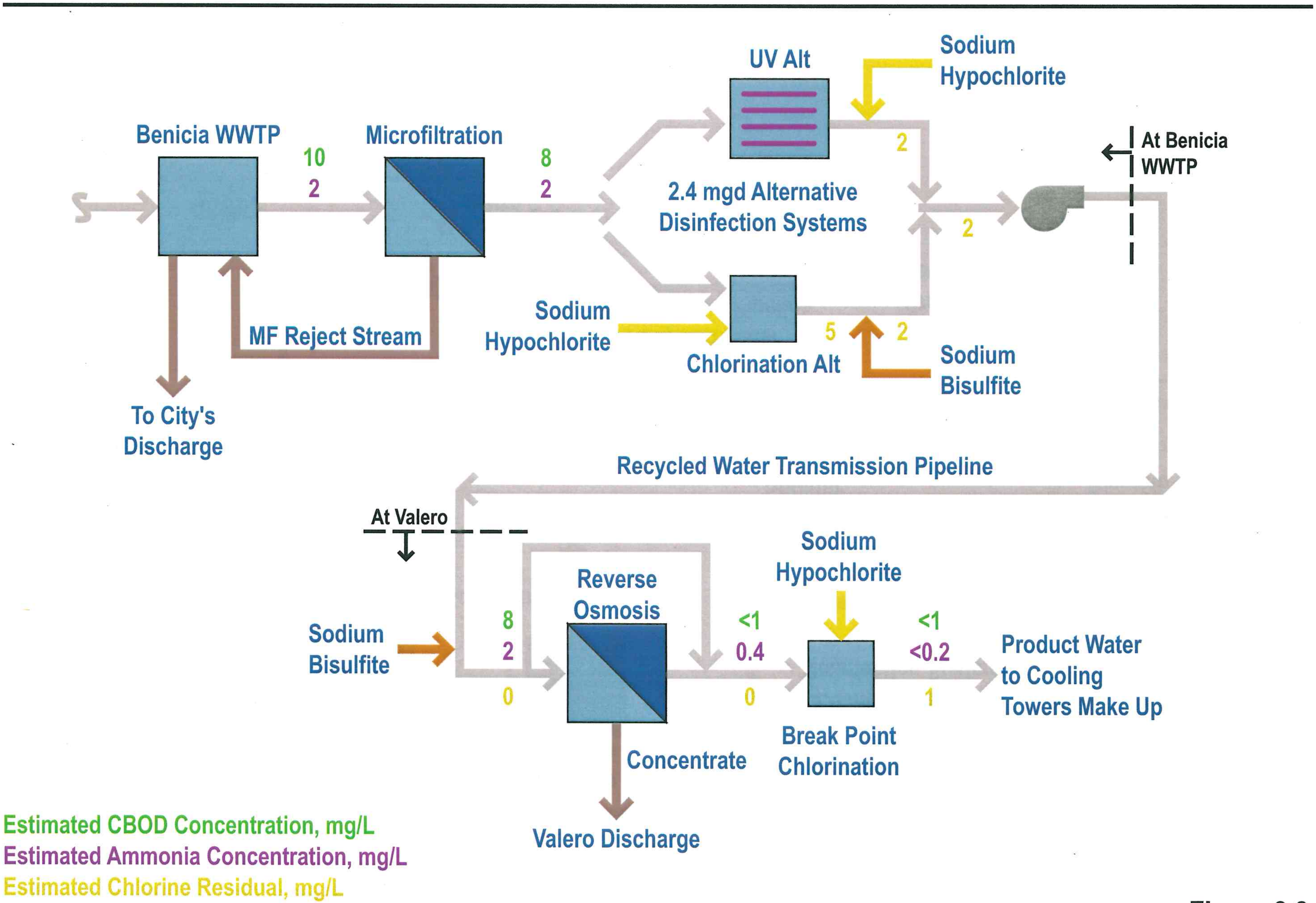

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Figure 3.3
Preliminary Process Block Diagram for 2.4 mgd Alternative Disinfection Systems

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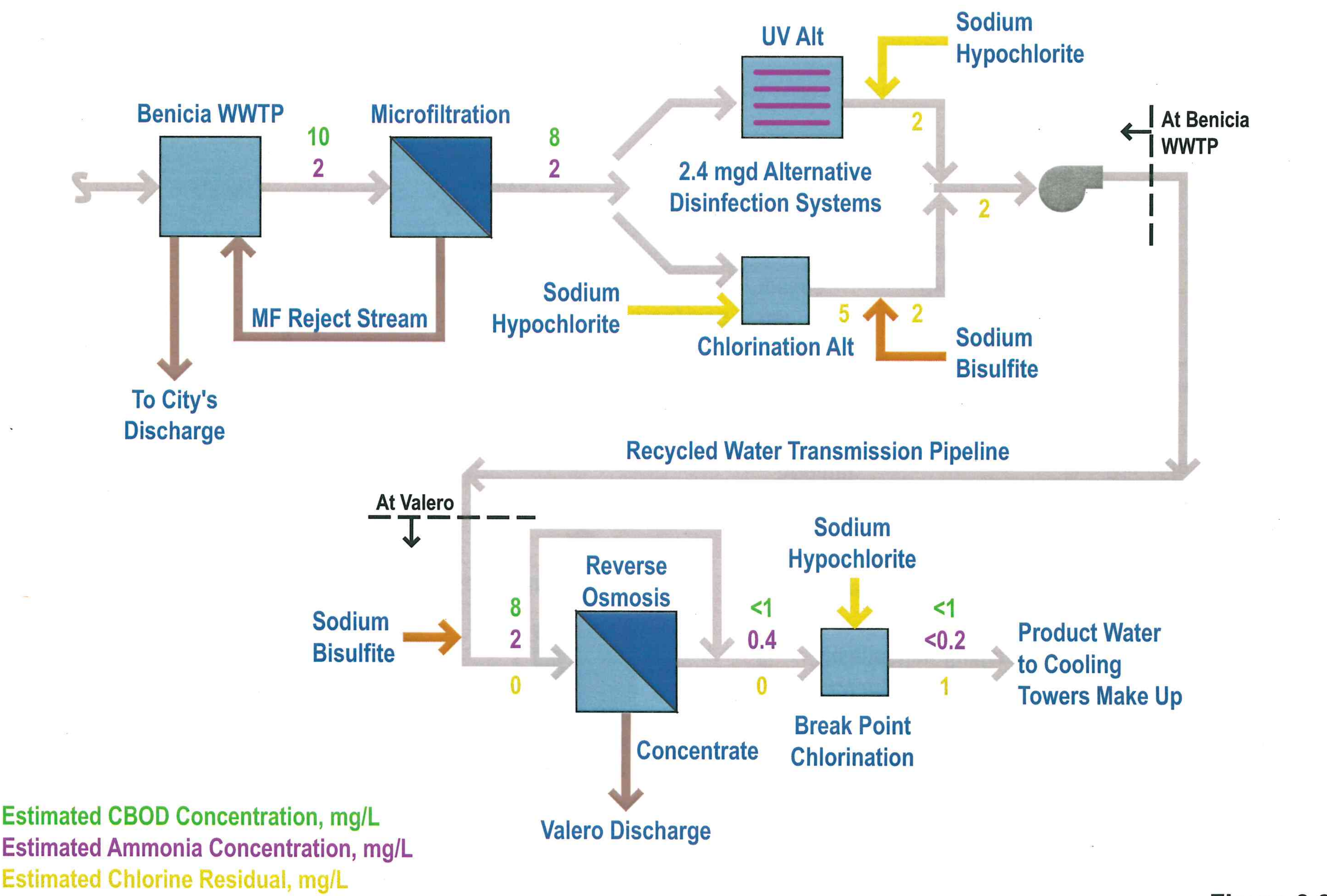

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Figure 3.3
Preliminary Process Block Diagram for 2.4 mgd Alternative Disinfection Systems

The City currently uses sodium hypochlorite to disinfect its secondary effluent. Hypochlorite would also be the chlorine source for the chlorination alternatives for the Water Reuse System. Hypochlorite is delivered to the plant site in tanker trucks and stored in bulk storage tanks. Metering pumps are used to inject sodium hypochlorite from the storage tanks into a rapid mix chamber upstream of the chlorine contact tank. The metering pumps are controlled based on flow and measured residual to maintain a minimum residual in the flow out of the tank.

A 12.5 percent hypochlorite solution is used for disinfection purposes, which is about twice the concentration of household bleach. Hypochlorite at this concentration is a relatively stable solution at normal temperatures and pressures. It is neither explosive nor flammable. However, it is a strong oxidizer and a severe irritant to skin and eyes on contact. Secondary containment is required for bulk storage facilities for potential leaks and spills.

Chlorine residual would be required in the recycled water pumped to the conveyance pipeline to Valero. However, too high of a residual could cause corrosion problems in the pipeline and at the cooling towers. Hence, dechlorination will be required to reduce the chlorine residual in both chlorination systems to an acceptable level. Dechlorination would be accomplished by adding sodium bisulfite to the flow stream to reduce the chlorine. The facilities required for bisulfite dechlorination would be similar to hypochlorite and would include bulk storage tanks, chemical metering pumps, and mixing equipment. Bisulfite has similar safety issues as hypochlorite and would require secondary containment. Bisulfite is also presently being used at the City's WWTP.

CT is a parameter used for design of chlorination systems and is the product of chlorine residual concentration and modal contact time. As discussed in Section 2, Title 22 requires a minimum CT of 450 milligram-minutes per liter at all times with a modal contact time of at least 90 minutes. As mentioned above, in order to insure that the 90 min modal time is achieved, a design hydraulic residence time of 120 min is typically used.

Adding hypochlorite and bisulfite to the process flow stream will increase the level of total dissolved solids (TDS) in the recycled water. However, given the relatively low projected TDS of the RO system, this should not cause any problems. However, for the alternative wherein chlorination follows MF and not RO, there would be design implications for the RO system. Since chlorination will add chlorides to the water, the RO system may have to be designed with a smaller split flow by pass than the 15% that is projected for the alternative which has chlorination following the RO process.

3.3 UV Disinfection Systems

There are basically three types of UV disinfection systems:

- Low Pressure, Low Intensity Lamps
- Medium Pressure, High Intensity Lamps
- Low Pressure, High Intensity Lamps

In recycled water applications these systems are generally all installed in open channels.

All of these systems use ultraviolet light to destroy the ability of pathogenic organisms to reproduce, thus eliminating their potential for infection. The design parameter for UV disinfection is dose, which is the product of UV intensity and exposure time. Types of UV systems are defined by the type of mercury vapor lamp used to produce the light and the configuration of the lamps used to apply the light to the process flow. The three types of systems, cited above, use different types of lamps arrayed either horizontally or vertically to the flow stream submerged in an open channel. Each UV lamp is enclosed in a sealed quartz sleeve to protect the lamp. The sleeves become fouled over time as compounds in the flow stream accumulate, limiting the amount of UV applied to the flow stream. Fouling increases as the lamp temperature increases with the higher powered lamps. Cleaning of the lamp sleeves is therefore an important consideration for UV systems.

UV provides no residual disinfectant. To prevent biological growth in the recycled water transmission pipeline, it is therefore necessary to add chlorine to the UV disinfected flow stream prior to pumping to the distribution system. This will require sodium hypochlorite storage and pumping facilities similar to those described above, except for a significantly lower dose and no chlorine contact tank.

As discussed in Section 2, any UV disinfection system proposed for Title 22 water recycling use must have prior conditional acceptance of the DHS under the 2003 NWRI Guidelines.

3.3.1 Low Pressure, Low Intensity UV Disinfection

Low pressure, low intensity (LPLI) UV was the first type of UV system commercially developed for municipal wastewater disinfection. Low pressure, low intensity UV lamps draw approximately as much power as household fluorescent lamps (70 to 80 watts). These types of lamps are basically monochromatic, producing UV light at a wavelength of 254 nanometers, very near the most germicidal wave length of 260 nanometers. Lamp temperature is 100 to 120 degrees Fahrenheit. Channel depth is typically about 4 feet, but will depend on the number of lamps used for each UV module. These systems have constant lamp power outputs compared to variable power outputs for the medium pressure, high intensity and low pressure, high intensity systems. Due to the number of lamps required, it is usually possible to effectively control system power output by controlling the number of lamp banks in operation. Generally, for systems with capacity of 2 mgd, most of the major UV equipment manufacturers do not market the LPLI equipment because it is usually not cost-effective. Therefore, LPLI UV was not considered for this application.

3.3.2 Medium Pressure, High Intensity UV Disinfection

Medium pressure, high intensity UV systems were developed to make UV disinfection more practical for larger wastewater flow rates and for wastewaters with high turbidities and low UV transmittance. Medium pressure lamps draw approximately 2,800 watts per lamp, and are polychromatic, producing UV light over a wider range of wavelengths than low pressure, low intensity lamps. Lamp temperature is typically several hundred degrees Fahrenheit. Channel depth typically varies from eight to 12 feet.

The advantages of medium pressure, high intensity (MPHI) systems are as follows:

- Approximately ten times fewer lamps are required than low pressure, low intensity and two to three times fewer than low pressure, high intensity for the same conditions.
- Automated wiping of the sleeves is practical due to the smaller number of lamps and larger spacing between lamps.
- Variable power output.

The disadvantages of the medium pressure, high intensity systems are:

- Due to the polychromatic nature and high temperature, it is the least energy efficient of the three lamp types.
- Due to the high temperature, the lamps foul at a higher rate than the other lamp types.
- The medium pressure lamps have a shorter lamp life than the other lamp types.
- Since only one manufacturer produces this equipment, selecting this system would result in sole source procurement.

Similar to the LPLI systems, the UV manufacturer (namely, Trojan) is not marketing the MPHI system for recycled water applications, because they are generally not competitive with Low Pressure High Intensity, particularly for the design flow of this Project. Hence, MPHI UV was not considered for this application.

3.3.3 Low Pressure, High Intensity UV

Low pressure, high intensity (LPHI) UV systems have been developed and implemented over the last five years. LPHI systems have the advantages of high intensity (although not as high as medium pressure) and automatic wiping systems without the reduced energy efficiency, shorter lamp life and increased fouling rate of the MPHI systems. Low pressure, high intensity lamps draw between 250 and 360 Watts per lamp depending on the manufacturer. These systems include automated mechanical wiping systems. Chemical cleaning by physically removing each array of lamps from the channel and placing it in a cleaning bath is required once or twice a year, according to one manufacturer. Channel water depth varies between about 2.5 ft and six feet.

Advantages of the low pressure, high intensity systems are as follows:

- Three to four times fewer lamps are required than for low pressure, low intensity for the same conditions.
- More energy efficient than medium pressure, high intensity systems. Nearly as efficient as low pressure, low intensity on a per lamp basis.
- Low temperature results in lower fouling rate than the MPHI systems.
- Potential for multiple manufacturers for competitive bidding.
- Variable power output.

Low pressure, high intensity UV systems approved by DHS for Title 22 applications under the 2003 NWRI Guidelines are:

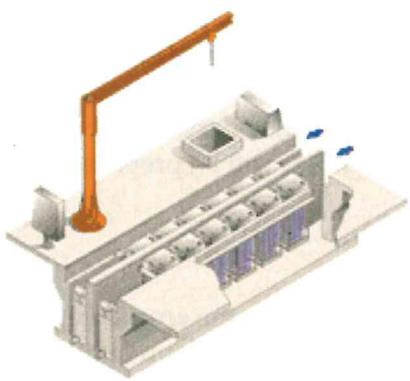
- Wedeco Environmental Technologies Specktrotherm TAK 55HP UV
- Ondeo IDI – Aquaray 40 HO VLS System
- Trojan Technologies - UV 3000Plus System

3.3.4 Summary of Alternative UV Systems

Table 3-2 presents a summary of the characteristics of the three UV systems described above. Hence, based on the qualitative analysis of the three systems, the experience of CDM on its other UV projects and the current marketing postures of the key equipment manufacturers in this field, the analysis of UV disinfection application to the Benicia Water Reuse Project will be based on only Low Pressure, High Intensity equipment. Figure 3.4 contains graphics of each of the three systems. As noted above, Trojan is the only serious manufacturer of MP-HI system for open channel contact.

Trojan UV 4000 Plus Medium Pressure High Intensity System – Typical Module

Wedeco UV Module – Typical for either Low Pressure Low Intensity or Low Pressure High Intensity Lamps



IDI-Ondeo UV Aquaray Module – Typical for either Low Pressure Low Intensity or Low Pressure High Intensity Lamps

Trojan UV 3000 Plus Low Pressure High Intensity System – Typical Module

CDM

Figure 3.4
Alternative UV Equipment

Table 3-2
Summary of Alternative UV System Characteristics

Parameter	LP-LI	MP-HI	LP-HI
Number of Lamps	Most	10% of LP-LI	25-30% of LP-LI
Power Draw per Lamp, Watts	80	2800	250 to 360
Cleaning System	Manual	Automatic	Automatic
Output Light Type	monochromatic	monochromatic	polychromatic
Lamp Life, hours	12,000	10,000	~ 10,000
Energy Efficiency	4x MP, HI	lowest	3x MP, HI
Variable Power Output	No	Yes	Yes
Potential Manufacturers	3	1	3
Feasibility for Benicia Water Reuse Project	No – Not cost-effective at 2 mgd size range	No – Not cost-effective at 2 mgd size range	Yes

4.0 Bases for Cost Estimates

Construction cost estimates presented herein have been developed for the purpose of comparing alternatives. Cost differences are important in that they help to distinguish the economics of one alternative over another. Actual construction and O&M costs can vary from the costs included herein. Common components have not been included. After all major project components are defined, estimated construction and O&M costs of the entire Water Reuse Project will be prepared and presented in the Conceptual Design Report.

4.1 Construction Cost Estimates

- **Foundations** – Owing to the poor soil conditions (Bay mud) in the area available for the Project, it will be necessary to place new structures on pile foundation systems. Based on review of the Geotechnical Engineering and Environmental Services Report, dated 15 July 1997 and prepared by Harza Engineers for the City's 1998 WWTP Improvement Project, pre-cast concrete piles, driven to an approximate depth of 70 feet have been assumed. Conceptual design estimates were made of the number of piles per structure, plus mobilization and demobilization. Pile driving costs were assumed at $40/foot of pile, including the cost of the pile. Estimates were based on budget quotations obtained from a local pile driving subcontractor.

- **Structural** – Chlorine contact tanks and UV channels were assumed to be constructed of cast-in-place reinforced concrete. Structural concrete costs for the chlorine contact tanks were estimated at $1.5/gallon capacity and UV channels were estimated at $3/gallon capacity because they are much smaller in capacity than chlorine tanks and require special interior coatings. Structural costs include excavation, reinforced concrete and structural backfill. Also included is aluminum handrail for uncovered tanks. The UV channels would be covered with aluminum checker plate, which was estimated at the unit cost of \$30/sf.

- **Civil** – Civil site work costs were estimated at 20% of structural costs (excluding foundation costs) to cover site preparation, grading, paving and site piping.

- **Mechanical –** Mechanical equipment costs were obtained from vendors and/or were based on experience from other similar projects. Budgetary costs for UV equipment were obtained from Trojan Technologies.

- **Electrical** – Power supply will be required to the UV system components and to chlorine mixers and feed pumps. Electrical costs for both the UV and the chlorination systems were estimated at 30 percent of the mechanical equipment cost based on experience with construction of similar systems.

- **Instrumentation** - Instrumentation will be required for process monitoring and control and for connection to the plant SCADA system. Typical instrumentation includes monitoring of water levels, chlorine residual, pH, UV transmittance, and others. Monitoring of data available from the manufacturer-furnished UV control panel will be provided. The instrumentation costs are estimated at 20 percent of mechanical equipment cost.

Contractor's overhead and profit are included at 15 percent. Owing to the level of detail developed in this conceptual design phase a contingency allowance of 25 percent is included to account for lack of detailed information, estimating inaccuracies, and relatively small items that may not have been included.

4.2 Operating and Maintenance Cost Estimates

- **Electrical Power Cost –** Electrical power costs used were \$0.12/kWhr, which is based on the average unit price for power at the WWTP for one winter month and one summer month.

- **Labor Cost –** Labor cost was assumed at \$50/hr, which includes City's normal general and administrative overhead, as well as holidays, sick leave, and vacation.

- **Chemicals –** Costs of sodium hypochlorite and sodium bisulfite were obtained from City WWTP staff for actual cost paid for these chemicals.

- **Other Consumables –** Cost of other consumables, such as UV lamps and ballasts were obtained from Trojan Technologies and are presented within the analysis of each system.

5.0 Conceptual Designs and Estimated Costs Of Chlorination Systems

5.1 Conceptual Design and System Sizing of Chlorination Systems

Conceptual designs were prepared for the two chlorination systems, using the design criteria in Table 3-1. Table 5-1 contains the design criteria, facility sizing and chemical demands. Figure 5.1 contains a potential location and footprint for the two chlorination systems. As can been seen in the figure, the chlorination process would take up a large amount of the available space for the Water Reuse Project.

Table 5-1
Design Criteria and System Sizing for Chlorination Systems

Parameter	Units	Values for 2.0 mgd System	Values for 2.4 mgd System
CT (Cl2 Residual x Contact Time)	mg-min/L	450	450
Chlorine Residual at end of Contact	mg/L	5	5
Modal Contact Time	min	90	90
HRT(1) at ADF	min	120	120
Assumed Tank Side Water Depth by Width	ft	10x10	10x10
Volume Required	gal	167,000	200,000
Volume Required	cf	22,300	26,700
Assumed Number of Bays	No.	2	2
Volume per Bay	cf	11,150	13,400
Total length of Bay	ft	112	134
Assumed Number of Passes Per Bay	No.	3	3
Length per Pass each Bay	ft	38	45
Total Footprint Area both Bays	sf	2,600	3,100
Estimated Chlorine Dose	mg/L	8	12
Assumed Chlorine demand	Mg/L	3	7
Estimated Sodium Hypochlorite Required(5)	gpd	133	240
Estimated Chlorine Residual After Contact Time	mg/L	5	5
Desired Final Chlorine Residual of Product Water for Pipeline Transmission	mg/L	2	2
Chlorine Residual to be Reduced	mg/L	3	3
Estimated Sodium Bisulfite Dose Required	mg/L	ni(4)	4.8(2)
Estimated Sodium Bisulfite Required	ppd	ni(4)	80
Estimated Sodium Bisulfite Required	gpd	ni(4)	52(3)

(1) HRT = Hydraulic Residence Time. In a well-designed chlorine contact tank, the actual modal time should equal or exceed approximately 0.75 * theoretical HRT.
(2) It takes 1.46 mg/L of sodium bisulfite to destroy 1.0 mg/L of chlorine residual. Supplier recommends using 1.6 mg/mg.
(3) At 25% commercial strength, 1 gallon of sodium bisulfite contains 2.5 pounds of active sodium bisulfite. (38% strength not recommended for this application by supplier, owing to freezing or precipitation.)
(4) ni = not included. Because dechlorination is common to both UV and chlorination for the 2.0 mgd alternatives, facilities and costs for dechlorination have not been included.
(5) At 12.5% solution one gallon of sodium hypochlorite equals one pound of chlorine.

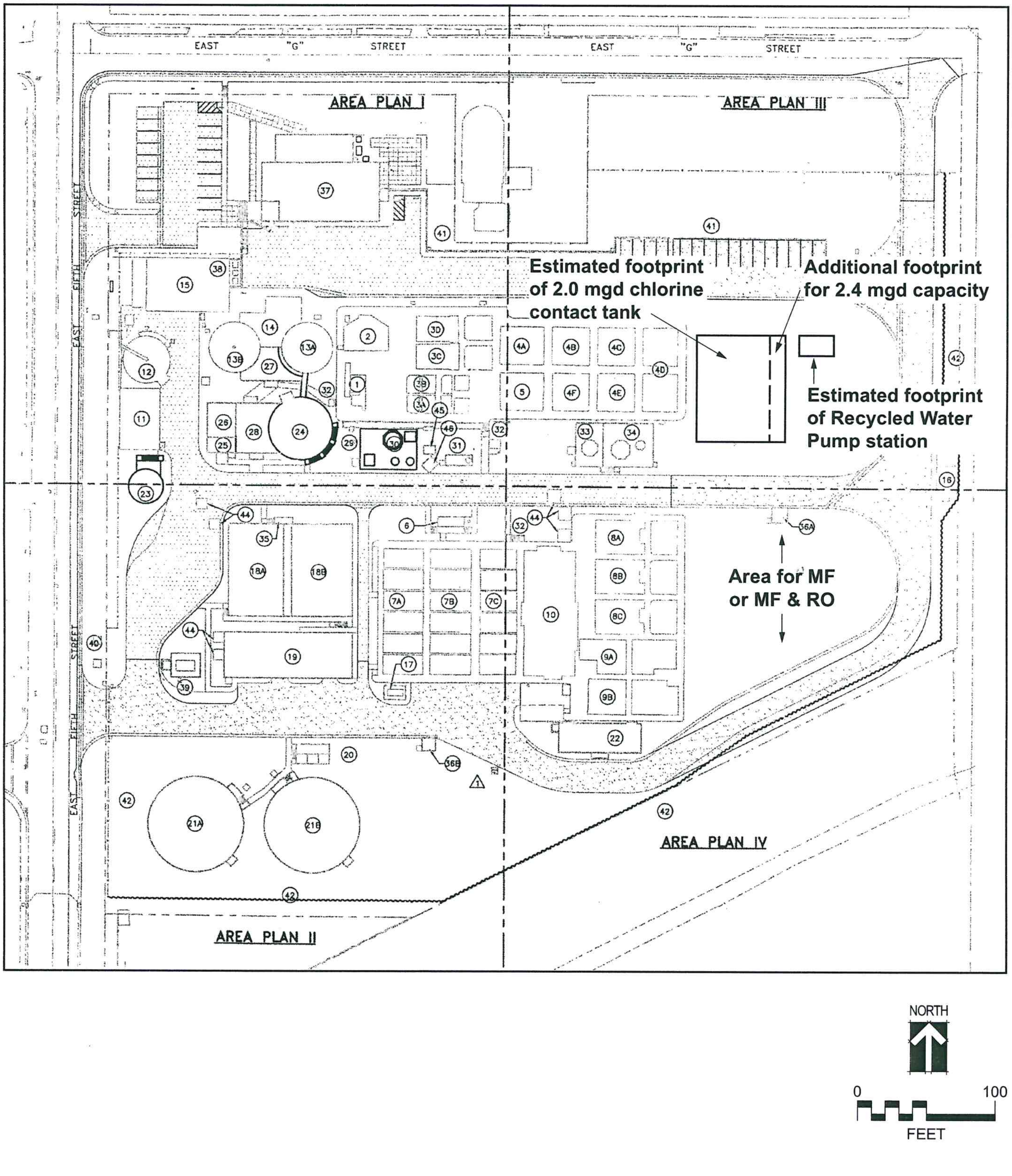


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Figure 5.1
Conceptual Site Plan for Chlorination Systems

For either chlorination alternative (2.0 or 2.4 mgd), an estimated chlorine residual of approximately 5 mg/L will remain in the water. This residual will need to be reduced to approximately 2 mg/L before pumping the water to Valero. This chlorine residual will prevent slime growth in the transport pipeline. Chlorine residual of 5 mg/L could negatively impact the lining of the transport pipeline. Under the 2.0 mgd alternative, chlorination would have to go through breakpoint in order to result in a chlorine residual of 5 mg/L to meet Title 22 requirements. Hence, sodium bisulfite would have to be added to reduce the residual down to 2 mg/L before pumping to Valero. (Please refer to Figure 3-2.)

Under the 2.4 mgd alternative, it is anticipated that approximately 2 mg/L ammonia will be present from the secondary treatment process. Adequate chlorine would be added to achieve a 5 mg/L residual without going through the breakpoint, which would require a very high dosage. Hence a design dose of 12 mg/L has been estimated (5 mg/L for the Title 22 requirement and 7 mg/L to account for other demands required for the formation of chloramines). (Please refer to Figure 3.3.) After chlorination, the residual will be reduced to 2 mg/L by adding sodium bisulfite. Once the recycled water reaches the Refinery, additional sodium bisulfite must be added to destroy any remaining chlorine residual because it is detrimental to the RO membranes.

5.2 Estimated Chlorination Construction Costs

Using the sizing of facilities for the two chlorination options from Table 5-1 and the unit prices and methodology discussed in Paragraph 4.1 above, construction cost estimates were prepared for the two systems and are presented in Table 5-2. As can been seen from the table, the 2.4 mgd system has 20% more capacity than the smaller system, and its estimated construction cost is similarly about 20% higher.

5.3 Estimated Annual Operations and Maintenance Requirements for Chlorination Systems

Based on the design criteria presented in Table 5-1, estimates of operating and maintenance requirements were developed, are presented in Table 5-3, and include chemicals, power, labor, and equipment replacement. Labor levels of effort are based on the experience of the staff at the City's WWTP for its existing chlorination system and include regular cleaning and adjusting the chlorine residual analyzers.

Table 5-2
Estimated Construction Cost of Chlorination Systems

Item	*Quantities*	*Unit Prices*	*2.0 mgd System Extensions $1,000's*	*2.4 mgd System Extensions $1,000's*
Contact Basins - Structural				
2.0 mgd system-volume, gal	167,000	$1.50/gal	$250	
2.4 mgd system-volume, gal	200,000	$1.50/gal		$300
Contact Basins – Civil/Site	LS	20% Struct	$50	$60
Contact Basins –Pile Foundations	LS		$150	$170
In-Line Chlorine Mixers (1 each Bay)	2	$10,000 ea	$20	$20
Chlorine Analyzers	2	$15,000 ea	$30	$30
Chlorine Residual Sample Pumps	2	$10,000 ea	$20	$20
Hypochlorite Feed Pumps	2	$15,000ea	$30	$30
Hypochlorite Storage	2,000	$5/gal	$10	$10
Sodium Bisulfite Feed Pumps	2	$10,000ea	ni(1)	$20
Sodium Bisulfite Storage	500 gal	$7/gal	ni(1)	$4
Miscellaneous Chem. Piping & Valves	Lot	Lump Sum	$50	$60
Subtotal			$610	$724
Add 50 % of Mech Equip for Elect & ICM			$75	$100
Subtotal			$685	$824
Add 25% Contingency			$170	$206
Subtotal			$855	$1,030
Add 15% Contractor OH & P			$125	$160
Total Estimated Construction Cost			**$980**	**$1,190**

(1) ni = not included. Refer to footnote 4 in Table 5-1

Table 5-3
Estimated Annual Operations and Maintenance Requirements for Alternative Chlorination Systems

O&M Item	*Dose/Units*	*Annual Quantities for 2.0 mgd System*	*Annual Quantities for 2.4 mgd System*
Hypochlorite	gal/yr	48,700	87,600
Power for One 2 HP Cl2 Feed Pump	kWhr/yr	11,000	11,000
Sodium Bisulfite	gal/yr	ni(2)	11,700
Power for One, 1 HP NaHSO3 Feed Pump	kWhr/yr	ni(2)	5,000
Gen Equip Replacement & Repair	3%/yr	$225,000 (base)	$280,000 (base)
O&M Labor(1)	15 hrs/wk	780	780

(1) Based on discussions with City's WWTP staff.
(2) ni = not included. Refer to footnote 4 in Table 5-1.

5.4 Estimated Annual Operation and Maintenance (O&M) Costs for Chlorination Systems

Using the information and assumption presented in Table 5-3, annual O&M costs were estimated for the two systems and are presented in Table 5-4. As can be seen from the costs presented in Table 5-4, the 2.4 mgd system is about 40% more costly in O&M than the smaller system. This is because the required chlorine dose is 50% higher, as explained in paragraph 5.1, above, and partial dechlorination is required for the 2.4 mgd system, but not for the 2.0 mgd system, as also explained in paragraph 5.1.

Regarding the dechlorination requirements for the 2.0 mgd alternatives, as noted in the footnotes for Tables 5-1 through 5-4, facilities and costs for feeding sodium bisulfite have not been included because they are common to both the UV and the chlorination alternatives for the 2.0 mgd size. Including facilities that are common to alternatives tends to mask economic differences between alternatives. As shown in Figure 3-2, dechlorination would be required for both the UV and chlorination and the equipment size and dose rates would be the same. This explains why these facilities are not included.

Table 5-4
Estimated Annual Operations and Maintenance Costs for Alternative Chlorination Systems

O&M Item	*Annual Quantities for 2.0 mgd System*	*Annual Quantities for 2.4 mgd System*	*Unit Prices*	*Estimated Annual O&M Cost for 2.0 mgd $1,000s/Yr*	*Estimated Annual O&M Cost for 2.4 mgd $1,000s/Yr*
Hypochlorite	48,700 gal	87,600 gal	$0.60/gal	$29	$53
Power for Cl2 Feed Pump	11,000 kWhr	11,000 kWhr	$0.12/kWHr	$2	$2
NaHSO3	ni(1)	11,700 gal	$0.60/gal	ni(1)	$7
Power for NaHSO3 Feed Pump	ni(1)	5,000 kWhr	$0.12/kWHr	ni(1)	$1
Gen Equip Replacement & Repair	$225,000	$280,000	3%/yr	$7	$8
O&M Labor	780 hr	780 hr	$50/Hr	$39	$39
Estimated Total Annual O&M Cost				**$77**	**$110**

(1) ni = not included. Refer to footnote 4 in Table 5-1.

6.0 Conceptual Designs and Estimated Costs Of UV Systems

6.1 Introduction

Conceptual designs of the two alternative UV disinfection systems have been developed for the purpose of economic and qualitative comparison. The UV system conceptual designs are based primarily on information provided by UV equipment manufacturers. This information establishes the number of lamps and lamp banks required to provide the design UV dose and the number and dimensions of UV channels. Design dose, redundancy requirements, and minimum number of banks per channel are in accordance with the requirements of the 2003 NWRI/AWWARF UV Guidelines.

The conceptual designs of the two alternative UV systems presented below are based on information provided by Trojan Technologies for its 3000Plus, Low Pressure, High Intensity UV disinfection system. Conceptual design information and budgetary costs were also obtained from, Wedeco and Ondeo, the other manufacturers of low pressure, high intensity UV disinfection systems. For the purposes of conceptual design and comparison, the Trojan information was used. Quotes from the other two vendors were comparable. Detailed evaluations of the three manufacturers would be performed in preliminary design, if UV is the finally selected disinfection process.

6.2 Conceptual Design and System Sizing of UV Systems

Based on the information presented in Sections 2 and 3, a summary of the general design criteria for the Low Pressure, High Intensity UV disinfection systems are presented in Table 6-1. Each of the proposed UV systems has a one channel layout with three banks in series including one standby bank. Figure 6.1 contains a footprint and potential location for UV systems in the space available for the Water Reuse Project at the City's WWTP site.

Breakpoint chlorination would be required for the 2.0 mgd system. Chlorine addition would be required for the 2.4 mgd system to provide a chlorine residual in the transport pipeline system. Table 6-2 presents a summary of the design criteria for the chlorination facilities required after the UV process.

Table 6-1
Conceptual Design of Low Pressure, High Intensity UV Disinfection Systems

Item Description	2.0 mgd System[1]	2.4 mgd System[2]
Number of Channels	1	1
Total Number of Banks	3	3
No. Redundant Banks	1	1
Modules per Bank	5	6
Lamps per Module	8	8
Total No. of Lamps	120	144
No. of Design Dose Lamps	80	96
No. of Redundant Lamps	40	48
Power Draw per Lamp	250 Watts	250 Watts
Max Power Draw Duty Lamps	20 kW	24 kW
Average Power Draw	17kW	20kW
Channel Dimensions, each		
Length[3], ft	75	75
Width, in	21	24
Channel Depth, in	60	60
Channel Volume, gal	5000	5700
Channel Surface Area, sf	130	150

[a] Reference Trojan Technologies UV3000Plus Proposal LJK1059C, dated 28 Sept 04.
[b] Reference Trojan Technologies UV3000Plus Proposal LJK1059B, dated 28 Sept 04.
[c] Includes additional length for future bank and inlet and outlet compartments

Table 6-2
Design Criteria and System Sizing for 2.0 mgd Break Point Chlorination System

Parameter	Units	Values
Chlorine Residual at end of Contact	mg/L	2[1]
HRT	min	10
Required Tank Volume	cf	1,900
Required Tank Volume	gal	14,000
Tank Dimensions: L x W x Side Water Depth	ft	12.5 x 12.5 x 12
Estimated Residual Ammonia	mg/L	0.4
Estimated Chlorine Dose[2]	mg/L	5
Estimated Sodium Hypochlorite Required[3]	gpd	83
Estimated Sodium Hypochlorite Required	Gal/yr	30,400

[1] Desired chlorine residual for prevention of slime growth and septicity in transport pipeline
[2] Chlorine dose is based on adding 3 mg/L for the breakpoint and 2 mg/L for final residual.
[3] At 12.5% solution one gallon of sodium hypochlorite equals one pound of chlorine

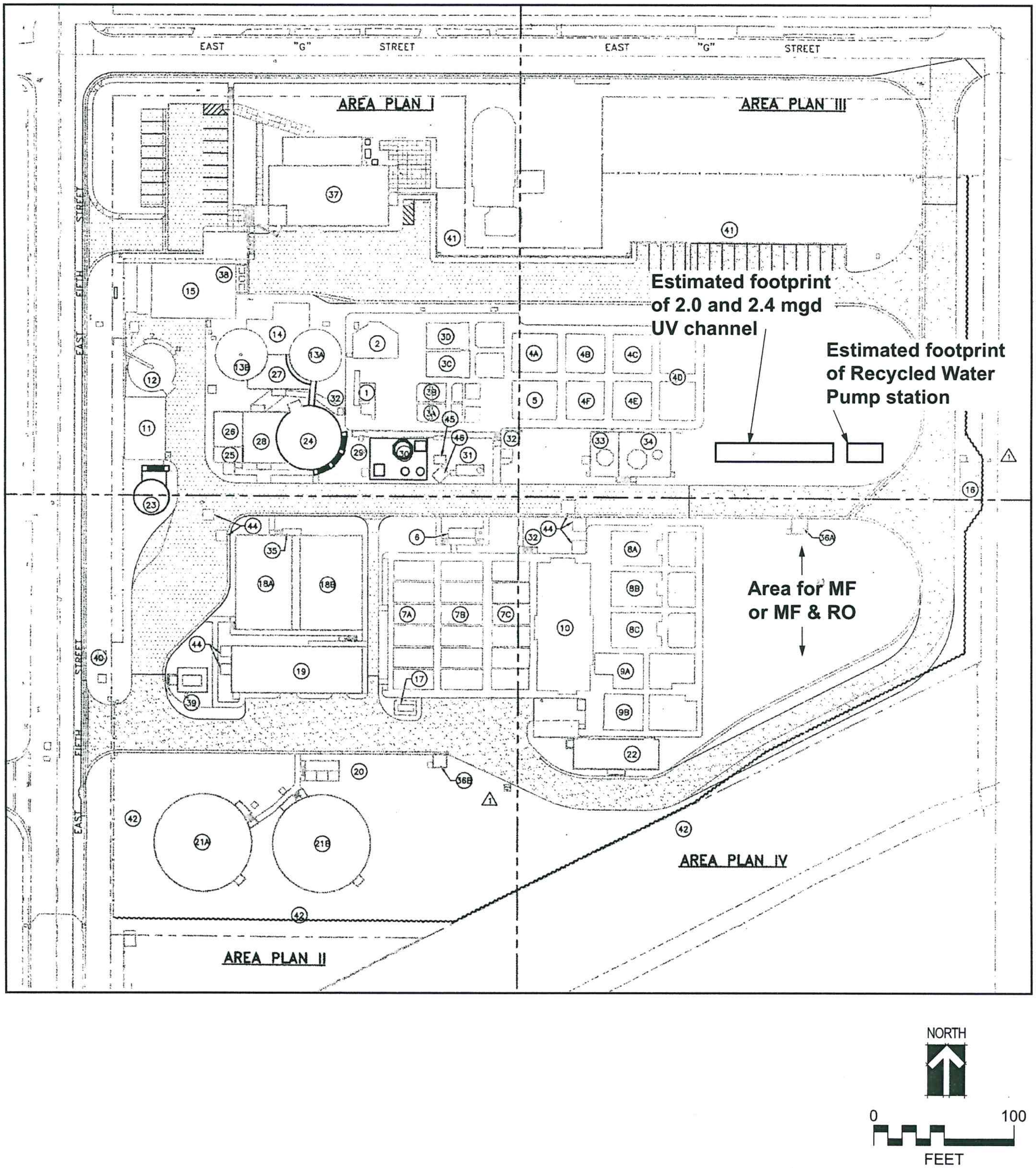

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Figure 6.1
Conceptual Site Plan for UV Systems

6.3 Estimated Construction Costs of UV Systems

Based on the unit prices and cost methodology, as presented in Section 4, and on the sizing of UV channels and required equipment, as presented in Table 6-1, construction cost estimates were prepared for the two UV alternatives. The equipment price quotes include all lamp banks, power distribution systems, control systems, and channel level controller. Table 6-3 presents the estimated construction costs for the two UV systems. As shown in the table, the 2.4 mgd system is approximately 11% more costly than the smaller system, although it has 20% higher capacity. This does not include costs for additional chlorination facilities.

Table 6-3
Estimated Construction Costs – Low Pressure, High Intensity UV Disinfection Systems

Items	*Quantities*	*Unit Prices ($/unit)*	*2.0 mgd System Extensions $1,000's*	*2.4 mgd System Extensions $1,000's*
UV Channels - Structural				
2.0 mgd Syst-volume, gal	5,000	$3/gallon	$15	
2.4 mgd Syst-volume, gal	5,700	$3/gallon		$17
Channels - Civil		20% Struct	$3	$3
Channels – Pile Foundation	LS		$40	$40
Covers for UV Channels				
2.0 mgd System-area, sf	130	$40/sf	$5	
2.4 mgd System-area, sf	150	$40/sf		$6
UV Equipment	LS		$310[1,3]	$350[2,3]
UV Equipment Installation	3 banks	$20,000/bank	$60	$60
Electrical/ICM (50% of Equip.)	LS		$155	$175
Subtotal			$590	$650
Add 25% Contingency			$150	$160
Subtotal			$740	$810
Add 15% Contractor OH & P			$110	$120
	Total Estimated Construction Costs		**$850**	**$930**

(1) Reference Trojan Technologies UV3000Plus Proposal LJK1059C, dated 28 Sept 04.
(2) Reference Trojan Technologies UV3000Plus Proposal LJK1059B, dated 28 Sept 04.
(3) Includes tax at 8.25%

6.4 Estimated Construction Costs for Chlorination Facilities Added to UV

Based on the sizing of chlorination facilities and required equipment, as presented in Table 6-2, construction cost estimates were prepared for the chlorination facilities required to be added to the two UV alternatives. Table 6-4 presents the estimated construction costs for these chlorination facilities.

Table 6-4
Estimated Construction Cost of Chlorination Facilities Required After UV Systems

Item	Quantities	Unit Prices	2.0 mgd System Extensions $1,000's	2.4 mgd System Extensions $1,000's
Contact Tank, Structural, vol.	14,000 gal	$1.5/gal	$20	na
Contact Tank, Civil		20% Struct	$4	na
Contact Tank, Foundation	LS		$25	na
Inline Chlorine Mixer	1	$10,000 ea	$10	$10
Chlorine Analyzer	1	$15,000 ea	$15	$15
Chlorine Residual Sample Pump	1	$10,000 ea	$10	$10
Hypochlorite Feed Pump	1	$10,000ea	$10	$10
Hypochlorite Storage	500	$6/gal	$3	$3
Miscellaneous Piping & Valves	Lot	Lump Sum	$20	$20
Subtotal			$117	$68
Add 50 % of Mech Equip for Elect & ICM	$65,000	50%	$33	$32
Subtotal			$150	$100
Add 25% Contingency			$40	$25
Subtotal			$190	$125
Add 15% Contractor OH & P			$30	$15
Total Estimated Construction Cost			**$220**	**$140**

6.5 Estimated Construction Cost of Combined UV and Additional Chlorination Facilities

Table 6-5 presents the combined estimated construction cost of the UV system and additional chlorination facilities. The estimated construction costs for the two UV systems are similar because the higher cost of UV equipment for the larger system is off-set by the breakpoint chlorination tank for the smaller, 2 mgd system.

Table 6-5
Combined Estimated Construction Costs of UV Systems with Chlorination Facilities

System Component	2.0 mgd UV System	2.4 mgd UV System
	$1,000's	**$1,000's**
UV Systems	$850	$930
Chlorination Facilities	$220	$140
Total Estimated Construction Costs	**$1,070**	**$1,070**

6.6 Estimated Operation and Maintenance Requirements for UV Systems

Estimated annual operations and maintenance (O&M) requirements were developed for each UV system. The O&M requirements are based upon the average annual flows of 2.0 mgd and 2.4 mgd, as discussed in Section 3, and include energy, lamp replacement, and labor for lamp replacement and cleaning.

- Estimates of power consumption were made based on the estimated connected load requirements and Trojan's estimated average annual power draw by the UV equipment for each system.

- Lamp replacement was based on estimates of lamp life information provided by Trojan.

- Ballast replacement is based on ballast failure percentage per year from information supplied by Trojan.

- Labor for lamp replacement is based on the estimate replacements required per year. Labor for manual cleaning is based on estimates of cleaning required per year considering experience at similar installations. It is noted that the proposed UV equipment of Low Pressure, High Intensity includes an automatic cleaning system for day to day cleaning. The Trojan system incorporates chemical application with its mechanical wiping system. The other two vendors do not have this feature.

Table 6-6 presents the estimated O&M requirements for the two UV systems.

6.7 Estimated Annual O&M Costs of UV Systems

Using the estimated O&M requirements presented in Table 6-6, estimates of annual O&M costs were developed using unit prices provided by Trojan and the labor rates from Section 4 above. Table 6-7 contains the estimated annual O&M costs for the two UV systems, including the chlorination requirements. The estimated O&M costs for the two UV systems vary by only 3%. This is due to the fact that the estimated cost of the higher chlorine dosage for the smaller system is off-set by the sum of higher power, labor and lamp replacement costs of the larger system.

Table 6-6 Estimated Operation and Maintenance Requirements for Low Pressure, High Intensity UV Disinfection Systems			
O&M Item	***Units***	***2.0 mgd System***	***2.4 mgd System***
Power			
Average Power Draw	kW	17[(a)]	20[(b)]
Annual Operating Hours	Hours	8,750	8,750
Annual Energy Use	kWhrs	150,000	180,000
Lamp Replacement			
Number of Duty Lamps		80[(1)]	96[(2)]
Lamp Life	Hours	10,000[(1)]	10,000[(2)]
Lamp Replacements	Number/year	70[(1)]	84[(2)]
Ballast Replacement			
Number of Ballasts		40	48
Failure Rate	%/yr	10%	10%
No of Ballast Replacements	No/yr	4	5
Chlorine Addition			
Estimated Dose	mg/L	5	3
Estimated Quantity	Gal/yr	30,400	21,900
Labor – UV			
Lamp Replacement	Hours/lamp	0.25	0.25
	Hrs/yr	18	21
Ballast Replacement	Hours/ballast	.25	.25
	Hrs/yr	1	1
Lamp Cleaning	Times/year	4	4
	Hours/Lamp	0.25	0.25
	Hrs/Yr	120	144
Instrument Cleaning and Process Monitoring	Hours/Week	5	5
	Hrs/Yr	260	260
Labor - Chlorination	Hours/Week	5	5
	Hrs/Yr	260	260
Total Estimated Labor Hours		659	686

(1) Reference Trojan Technologies UV3000Plus Proposal LJK1059C, dated 28 Sept 04

(2) Reference Trojan Technologies UV3000Plus Proposal LJK1059B, dated 28 Sept 04

Table 6-7
Estimated Operation and Maintenance Costs for UV Disinfection Systems

O&M Item	2.0 mgd System	2.4 mgd System
Energy		
Annual Energy Use(1)	150,000 kWhrs/Yr	180,000 kWhrs/Yr
Unit Cost of Energy(2)	$0.12/kWhr	$0.12/kWhr
Annual Energy Cost	$18,000	$21,600
Lamp Replacement		
Lamps per Year(1)	70	84
Unit Replacement Cost	$190/lamp(2,4)	$190/lamp(3,4)
Annual Lamp Replacement Cost	$13,300	$16,000
Ballast Replacement		
Ballasts per Year	4	5
Unit Replacement Cost(4,5)	$600	$600
Annual Ballast Replacement Cost	$2,400	$3,000
Chlorine Addition		
Annual Chlorine Usage	30,400 gal/yr	21,900 gal/yr
Estimated Unit Cost	$0.60/gal	$0.60/gal
Estimated Annual Cost	$18,200	$13,100
Labor		
Annual Labor	659 hours	686 hours
Labor Rate	$50/hour	$50/hour
Annual Labor Cost	$33,000	$34,300
Total Annual O&M Cost	**$84,900**	**$88,000**

(1) From Table 6-3
(2) Reference Trojan Technologies UV3000Plus Proposal LJK1059C, dated 28 Sept 04
(3) Reference Trojan Technologies UV3000Plus Proposal LJK1059B, dated 28 Sept 04
(4) Includes tax at 8.25%
(5) Estimated ballast cost provided by D.L Frost, Equipment Representatives for Trojan.

7.0 Quantitative Evaluation of Alternatives

7.1 Capital Cost Estimates

The capital cost of a project includes both the initial construction cost plus all "soft costs" that are required to implement the project. These costs include: engineering, construction management, administration, environmental compliance, acquisition of permits and financing costs. An amount of 25% of the estimated construction cost has been added to account for these soft costs.

7.2 Life Cycle Cost Analysis

The capital and annual O&M cost estimates presented herein are for comparative purposes only. These cost estimates are used to determine which type of disinfection technology is the most cost-effective in relation to each other. More detailed construction and O&M cost estimates will be developed for the selected alternative as part of the preliminary design.

Using estimated capital and annual O&M costs for each alternative system, present worth values were developed to compare the life-cycle costs of the two alternatives. Present worth is defined as that amount of money it takes to fund the capital investment of a project, as well as its annual operating and maintenance costs, over a period of time, given the cost of money (interest) during the evaluation period. For this analysis, the time period used was 20 years and the interest rate was six percent. Table 7-1 presents the results of this analysis.

Table 7-1
Summary of Present Worth Analysis for Alternative Disinfection Systems

	Alternative Disinfection Systems			
	2.0 mgd CL	***2.0 mgd UV***	***2.4 mgd CL***	***2.4 mgd UV***
	$1,000s[(1)]	***$1,000s[(1)]***	***$1,000s[(1)]***	***$1,000s[(1)]***
Estimated Construction Costs[(2)]	$980	$1,070	$1,190	$1,070
25% Allowance for Engineering, Admin and Legal Costs	$250	$270	$300	$270
Total Estimated Capital Costs	$1,230	$1,340	$1,490	$1,340
Estimated Annual O&M Costs[(3)]	$77	$85	$110	$88
PW of O&M Costs[(4)]	$880	$970	$1,260	$1,010
Total Estimated Present Worth[(5)]	**$2,110**	**$2,310**	**$2,750**	**$2,350**

(1) All Values have been rounded to the closest $10,000
(2) See Tables 5-2 and 6-2 for chlorine and UV estimated construction costs, respectively
(3) See Tables 5-4 and 6-4 for chlorine and UV estimated O&M costs, respectively
(4) O&M Cost times Present Worth Factor for 20 years at 6% interest. PWF=11.47.
(5) Equals the sum of Estimated Capital Cost and PW of Estimated O&M Cost

A review of Table 7-1 shows that for the 2.0 mgd systems, the PW of UV is approximately 9% higher than the chlorination system. The capital cost difference relates primarily to the breakpoint contact tank added to the UV system. O&M cost differences relate primarily to higher power cost for UV compared with higher chemical costs for chlorination. The 9% difference between the present worth of these alternatives is not within the accuracy of the conceptual costs estimates; hence, there is not clear distinction in cost-effectiveness between the two, 2 mgd alternatives.

However, for the 2.4 mgd systems, chlorination is approximately 17% higher than UV. This disparity is not surprising given the large structural investment required, which more than off-sets the cost of the UV equipment. Also, since the site requires pile foundations for water bearing structures, this increases the overall structural and civil costs. Also, chlorine addition is required to provide a chlorine residual in the transport pipeline. Hence, additional chemical facilities and operating costs are required in addition to UV facilities for the 2.4 mgd alternative (please refer to Figure 3.2). The 17% difference between the present worth of these alternatives is not within the accuracy of the conceptual costs estimates. However, there is a definite economic bias toward the UV system over chlorination for the 2.4 mgd alternatives.

7.3 Energy Cost Sensitivity Analysis

Considering that there is a significant difference in energy requirements for the two disinfection alternatives, it is appropriate to determine the impact that increases in power costs could have on the economic comparison of the two systems. If the cost of power were to increase to $0.16 per kWHr instead of the estimated $0.12 per kWhr, the present worth of the larger UV system would increase by approximately $80,000 to $2.46 million while the PW of the larger chlorination system would increase by only $8,000 to $2.23 million. This increase in energy costs does not significantly change the difference in present worth values between chlorination and UV regardless of system size in the size range evaluated. A drop in power cost would only widen the gap between the 2.4 mgd alternatives in favor of the UV system; it would narrow the gap between the 2.0 mgd alternatives, lessening the bias toward chlorination.

8.0 Qualitative Evaluation of Alternatives

In addition to capital cost, operating costs and overall present worth, it is appropriate to evaluate other qualitative factors to aid in the decision making process. Below is a discussion of pertinent qualitative factors. Table 8-1 contains a tabular summary of these discussions.

Table 8-1
Summary of Qualitative Evaluation of Chlorination and UV Disinfection

Qualitative Factors	*Chlorination*	*UV Disinfection*
Impact on Existing Facilities	High	Slight
Ease of Operation	Moderate	Moderate
Flexibility for Changing Requirements	Low	Good
Ease of Implementation	Moderate	Good
Future Expandability	Low	Good
Equipment Reliability	Good	Good
Process Reliability	Variable	Good
Proven Technology	High	Moderate
Process Complexity	Moderate	High
Impacts on Cooling Water Quality	Adverse	None
Safety	Adequate	Good
Public Acceptance	Adequate	Good

- **Impact on Existing Facilities:** Chlorination has a large footprint and has a major impact to the site. UV consumes much less available site area.

- **Ease of Operation:** Both systems are relatively easy to operate, although chlorination requires quite a bit of attention for maintaining the chlorine residual analyzers and chemical metering pumps. Depending on the effectiveness of the UV lamp cleaning systems, some manual cleaning of lamps may be required.

- **Flexibility to meet Changing Permit Requirements:** Although unlikely, the only permit requirement that would affect the disinfection process would be an increase in the virus inactivation requirement. UV would be able to respond to higher levels of inactivation much more easily than chlorination.

- **Ease of Implementation:** Both systems are relatively easy to implement. Regarding UV, it may be advantageous to pre-purchase the UV equipment. This would require some additional coordination during design and construction.

- **Future Expandability:** The UV system is more readily expandable and easier to phase, owing to the smaller contact tank and ease of adding more modules. Expanding the chlorine contact tank will consume additional footprint.

- **Equipment Reliability:** The equipment associated with chlorination has been used for many years and is well proven, even though residual analyzers require regular maintenance to insure reliable process readings. On the other hand, the UV equipment is still in the early application and development stages. The low pressure, high intensity system is relatively new. The vendors are still working on improvements to the lamp cleaning systems and the electrical ballasts. Standby, redundant equipment will be incorporated into the design of whichever system is selected and has been included in this analysis.

- **Process Reliability:** From a process reliability standpoint, the UV system has an edge. Chlorination is subject to the influence of chemical constituents in the water, namely any residual ammonia, nitrites and organic nitrogen. The Project also includes improvements to include nitrification facilities upstream of the Water Reuse Treatment System. It should be noted that the upstream biological process must be operated to complete the nitrification process. A partially nitrified effluent will contain residual nitrites, which exert a chlorine demand of about 7 to 1. Trace amounts of ammonia will consume chlorine through the breakpoint reaction. Residual organics will consume chlorine and form non-germicidal organo-chloramines. A number of plants that fully nitrify occasionally experience difficulty controlling the chlorination process. This situation can result in coliform excursions.

- **Proven Technology:** As stated above, chlorination has a much longer and proven track record than UV. However, there are now several successfully operating UV systems for the disinfection of recycled water for unrestricted use.

- **Process Complexity:** Both UV and chlorination are relatively complex from a process control standpoint. Regarding UV, the 2003 NWRI/AWWARF Guidelines require continuous monitoring of UV transmittance (UVT), UV intensity, turbidity and UV operational dose. As with chlorination, the UV process must automatically respond to changing water quality conditions. Also, it will be necessary to add chlorine, using sodium hypochlorite, and will need to employ online chlorine residual analysis. This is necessary to maintain water quality in the transmission system to Valero and to avoid odors. So some of the process complexities associated with chlorination are carried over to the UV system as well.

- **Power Demand:** UV requires substantially more power than chlorination. Therefore, operating costs are highly dependent on power rates. Generally speaking, chemical production costs fluctuate with power costs as well. Increases in chlorine costs would swing the analysis more in favor of UV.

- **Impacts on Water Quality for Recycle Purposes:** Chlorination adds chlorides and sulfates (from dechlorination with sodium bisulfite) to the water, which is adverse to cooling water requirements. UV adds no salts. However, in order to maintain a small residual in the recycled water transmission system to prevent slime and algal growth, a small amount of sodium hypochlorite will be added, which would also add a small amount of salts (sodium and chloride) to the recycled water.

- **Safety:** UV is a much safer system to operate. No Hazardous Material Maintenance and Management Plan is required; nor is a Spill Prevention and Containment Plan.

- **Public Acceptance:** Owing to the safety issue, public acceptance is perceived much higher for UV than for chlorination, even with sodium hypochlorite.

9.0 Conclusions and Recommendations

Based on the conceptual designs and economic analysis presented herein, the following conclusions are drawn:

- UV and chlorination appear to be nearly equally cost-effective for the two sizes of systems evaluated, namely 2.0 mgd and 2.4 mgd, given the accuracy of the conceptual estimates upon which they are based.

- Qualitative factors, in particular water quality impacts, site impacts and ease of process control, favor UV over chlorination.

Based on the above conclusions, CDM recommends that the City select the low pressure, high intensity UV system as the preferred disinfection system for the Benicia – Valero Water Reuse Project.

10.0 Use of UV Disinfection for Recycled Water Production

UV disinfection is in use for the production of Title 22 recycled water for unrestricted use applications in several locations in the San Francisco Bay Area. These publicly-owned recycled water plants include:

- Dublin San Ramon Services District (3 mgd)
- City of Santa Rosa (~20 mgd)
- City of Scotts Valley (1.5 mgd)
- City of Livermore (2 mgd)
- Mt. View Sanitary District
- American Canyon

Central Contra Costa Sanitary District also has UV disinfection, but it is for disposal and its effluent bacteria requirements are not as stringent as Title 22.

City of Benicia-Water Reuse Project Draft Technical Memorandum No. 3 – Recycled Water Conveyance System

To: *Chris Tomasik*

CC: *PURE Members*

DATE: *November 9, 2004*

Executive Summary

The purpose of this TM is to prepare a conceptual design of a complete conveyance system for recycled water from the Benicia WWTP to the Valero Refinery Cooling Towers. Rehabilitation of Valero's existing pipeline compared with construction of a new pipeline is evaluated.

Overview of the Recycled Water Conveyance System

The conveyance system will consist of a pump station at the City of Benicia WWTP, a pipeline approximately 14,000 feet in length and possibly a storage facility at the Refinery. Beginning at the WWTP the pipeline will travel from a new, high-lift pump station to the Valero "off site" dock line right-of-way in the vicinity of East 7th Street and "I" Street. The pipeline will follow the abandoned Valero dock lines northerly for about 9,000 feet to the Refinery property line. Within the Refinery the pipeline will follow Avenue "E" South, then up a vertical rise (known as a "waterfall") to Avenue "F" to the cooling towers.

The existing Valero dock lines are attached to above-grade structural steel frames, known as "sleepers." For major portions of the off-site alignment, an existing dock line could be used. However, along this alignment there are gaps where the dock line has been removed. At those gap locations, new pipe would be required. An evaluation of rehabilitating existing dock lines compared to constructing new piping is presented below. Within the refinery, new pipeline will be constructed on vertical extensions to the existing pipeline "sleepers" that parallel Avenues "E" and "F." A flow equalizing storage tank may be required near the cooling towers.

Alternative Flow Criteria

There are two flow criteria for the conveyance system, depending on where the RO system is located. For the scenario in which the RO system is located at the Benicia WWTP, the design capacity would be 2.0 mgd. If the RO system is located at the Refinery, the conveyance system needs to have adequate capacity to allow for the rejected concentrate flow of approximately 0.4 mgd from the RO membranes in order to produce 2.0 mgd of recycled water. For that scenario the design capacity of the system would need to be approximately 2.4 mgd.

Overview of Conveyance Pipeline Profile

Valero provided CDM with copies of plan and profile drawings of the "off-site" dock lines as well as information about the pipe material, pressure class, and wall thickness. Valero also provided information on the elevation of the existing pipeline sleepers within the Refinery. Using this information, CDM developed a preliminary profile of the pipeline from the City's WWTP to the cooling towers. The profile begins at the City's WWTP near Elevation Zero and reaches a high point approximately one mile northerly along the alignment at approximately Elevation 201. The pump station at the WWTP will be located at approximately Elevation Zero. Hence, the static lift under either of the flow scenarios will be about 200 ft.

Alternative Rehabilitation Pipe Lining Systems

Six types of lining systems were reviewed to determine their applicability to the existing Valero Dock Lines and to obtain conceptual unit costs. These lining systems basically break down into two generic types: bag or pipe-type prefabricated liners that are inserted into the host pipe; and, internally applied materials. These systems are described below.

Prefabricated Inserted Liners

- Fold and Form Liner
- Cured-In-Place Pipe
- Duraliner
- Swage Lining

Internally Applied Material Liners

- Epoxy Lining
- In-Situ Cement Mortar Lining

Based on the characteristics of the liners reviewed, Duraliner and Swage lining are the preferred alternatives because they are able to withstand the reclaimed water system pressure, do not require extensive interior surface preparation, and can accommodate abrupt changes in alignment. Both of these lining methods have similar estimated unit costs per foot, as well as similar access point requirements. Total unit construction cost of lining was estimated at $80 per lineal foot of pipe.

Alternatives to Rehabilitate or Replace Existing Valero Pipelines in Off-Site Right-of-Way

There are basically two alternatives for the segment of the pipeline from the lower end of the dock lines to a location approximately 2,000 feet south of the refinery property line and entrance road. Those two alternatives are:

- Alternative No. 1: Constructing a new 14-inch diameter pipe, mounted on the existing sleepers.
- Alternative No. 2: Rehabilitating the existing dock line segments that are available in the reach and providing new, 14-inch diameter pipe in locations where there are gaps.

The overall length of this evaluation is approximately 6,600 feet. Alternative No. 1 is composed of 6,600 feet of new 14-inch diameter pipe. Alternative No. 2 is composed of 5,800 feet of rehabilitated existing 12-inch diameter pipe plus 770 feet of new 14-inch diameter pipe to fill in the gaps in the existing available dock line. On both a first cost basis as well as a present worth basis, Alternative No. 1 is more cost-effective. Given the results of the economic analysis and the uncertainties and risks associated with rehabilitation, CDM recommends that the conveyance pipeline be constructed of new, 14-inch diameter cement mortar lined, steel pipe.

Summary of Estimated Construction Cost of Benicia-to-Valero Recycled Water Conveyance System

The Recycled Water Conveyance System consists of two capacity scenarios, as follows:

- 2.0 mgd capacity, if the RO System is located at the City's WWTP
- 2.4 mgd capacity, if the RO System is located at the Valero Refinery

Table ES-1 presents a summary of the estimated construction costs for the entire Recycled Water Conveyance System.

Table ES-1
Summary of Estimated Construction Costs of Recycled Water Conveyance System

System Component	***2.0 mgd Scenario***	***2.4 mgd Scenario***
	$1,000's	***$1,000's***
Recycled Water Pipeline(1)	$2,060	$2,060
Recycled Water Supply Pump Station	$490	$520
Total Estimated Construction Costs	**$2,550**	**$2,580**

(1) Estimated cost for the Pipeline Component does not include analysis or rehabilitation of existing sleepers.

Recommendation

Based on the analyses presented herein, CDM recommends that the City and PURE accept the recommendation to use a new, 14-inch pipeline for the conveyance system.

1.0 Introduction and Purpose of the Technical Memorandum

A joint Water Reuse Project is being undertaken by the City of Benicia and the Valero Refinery to supply approximately 2 mgd of recycled water for cooling water make-up at the Refinery.

TM 1, dated September 2004, evaluated alternative treatment processes to meet Valero's cooling water mineral requirements. The results of that evaluation were that the MF/RO process is the applicable water reuse treatment system to meet Valero's requirements. TM 2, dated 20 October 2004, evaluated alternative technologies for disinfection of recycled water produced by the micro-filtration and reverse osmosis process. The results of that evaluation were that the low-pressure, high-intensity UV process is the best disinfection system to meet regulatory requirements and Valero's water quality requirements.

The purpose of this TM is to prepare a conceptual design of a complete conveyance system for recycled water from the Benicia WWTP to the Valero Refinery Cooling Towers. Rehabilitation of Valero's existing pipeline compared with construction of a new pipeline is evaluated for the off-site alignment. This TM is composed of the following major sections:

- Overview of the Conveyance System
- Alternative Flow Criteria
- Conveyance Pipeline Profile
- Description of Existing Valero Off-Site Pipelines
- Recycled Water Quality Considerations for Pipeline Design
- Testing and Internal Inspection Methods for Existing Pipelines
- Description and Evaluation of Alternative Pipe Lining Systems
- Alternative Materials for New Pipe
- Alternatives to Rehabilitate or Replace Existing Valero Pipelines in Off-Site Right-of-Way

- Conceptual Design of Recycled Water Conveyance Pipeline
- Conceptual Design of Recycled Water Supply Pump Station
- Summary of Estimated Costs of Benicia-to-Valero Recycled Water Conveyance System
- Recommendation

2.0 Overview of the Conveyance System

The conveyance system will consist of a pump station at the City of Benicia WWTP, a pipeline approximately 14,000 feet in length and possibly a storage facility at the Refinery. Beginning at the WWTP, a new pipeline is required from a new, high-lift pump station to the Valero "off site" dock line right-of-way in the vicinity of East 7th Street and "I" Street. The pipeline will follow the abandoned Valero dock lines ("DL" which formerly carried petroleum products) northerly for about 9,000 feet to the Refinery property line. Within the Refinery the pipeline will follow Avenue "E" South, then rise vertically (called a "waterfall" by the refinery) up to Avenue "F", and then to the cooling towers. Figure 2.1 shows the proposed alignment superimposed on an aerial photo.

Within the Valero off-site ROW, the existing dock lines are attached to above-grade sleepers. For a major length of this portion of the alignment, an existing dock line could be used. An evaluation of rehabilitating existing dock lines compared to constructing new piping is presented below. For those portions of the off-site alignment that do not have available dock lines, new piping will be constructed. Within the Refinery, new pipeline will be constructed on vertical extensions to the existing pipeline "sleepers" that parallel Avenues "E" and "F."

A flow equalizing storage tank may be required near the cooling towers.

3.0 Alternative Flow Criteria

There are two flow criteria for the conveyance system, depending on where the RO system is located. Figure 3.1 shows a schematic diagram of the overall project with the RO system located at the Benicia WWTP. Under that scenario, the design capacity would be 2.0 mgd.

If the RO is located at the Refinery, the conveyance system needs to have adequate capacity to allow for the rejection rate of the RO membranes in order to produce 2.0 mgd of permeate, as shown in Figure 3.2. For that scenario the design capacity of the system would need to be approximately 2.4 mgd.

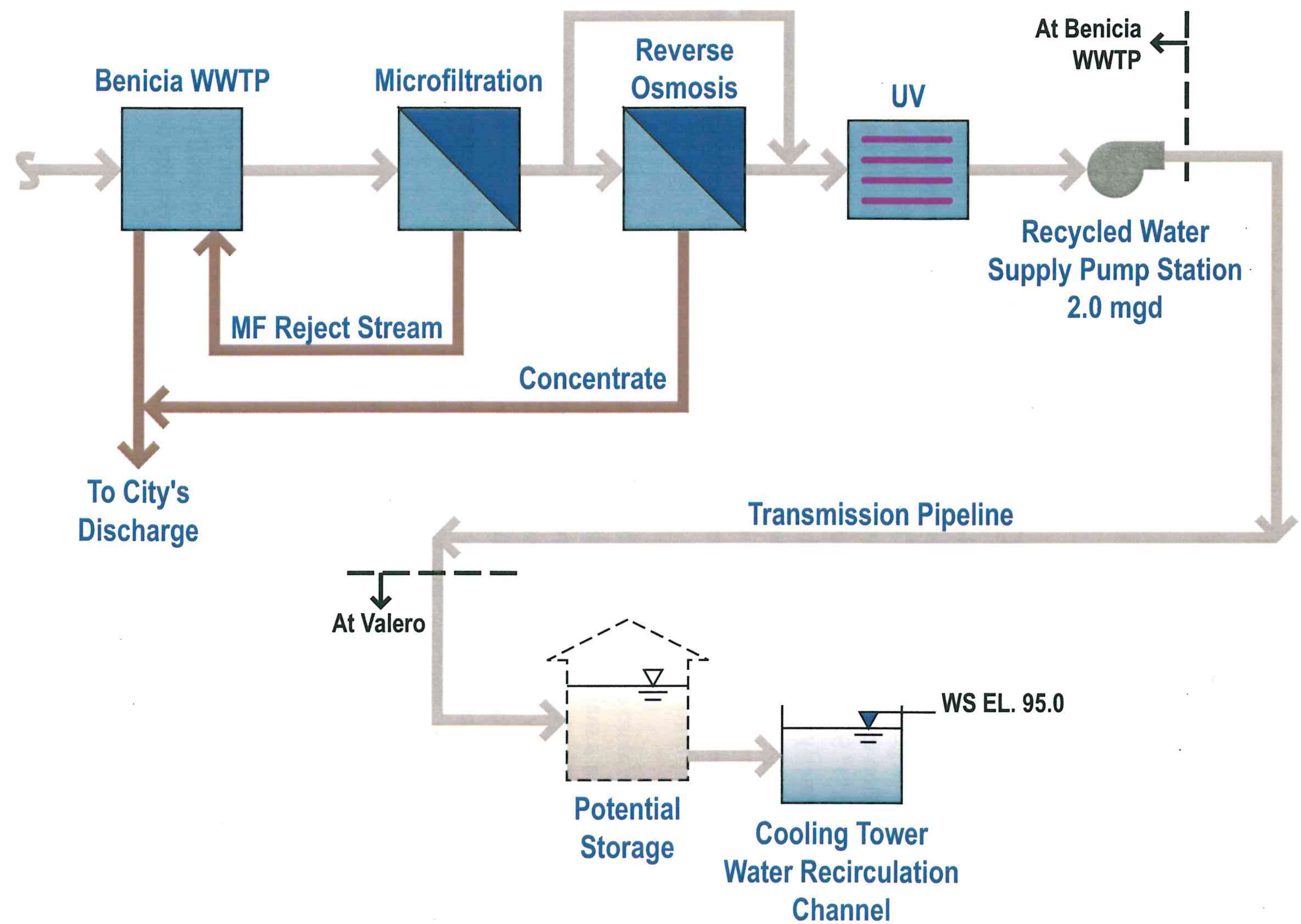

Figure 3.1
Benicia Water Reuse Project
Process Schematic for 2.0 mgd Recycled Water Supply Pump Station

CDM

Figure 2.1
City of Benicia Water Reuse Project Overall Site & Alignment Map

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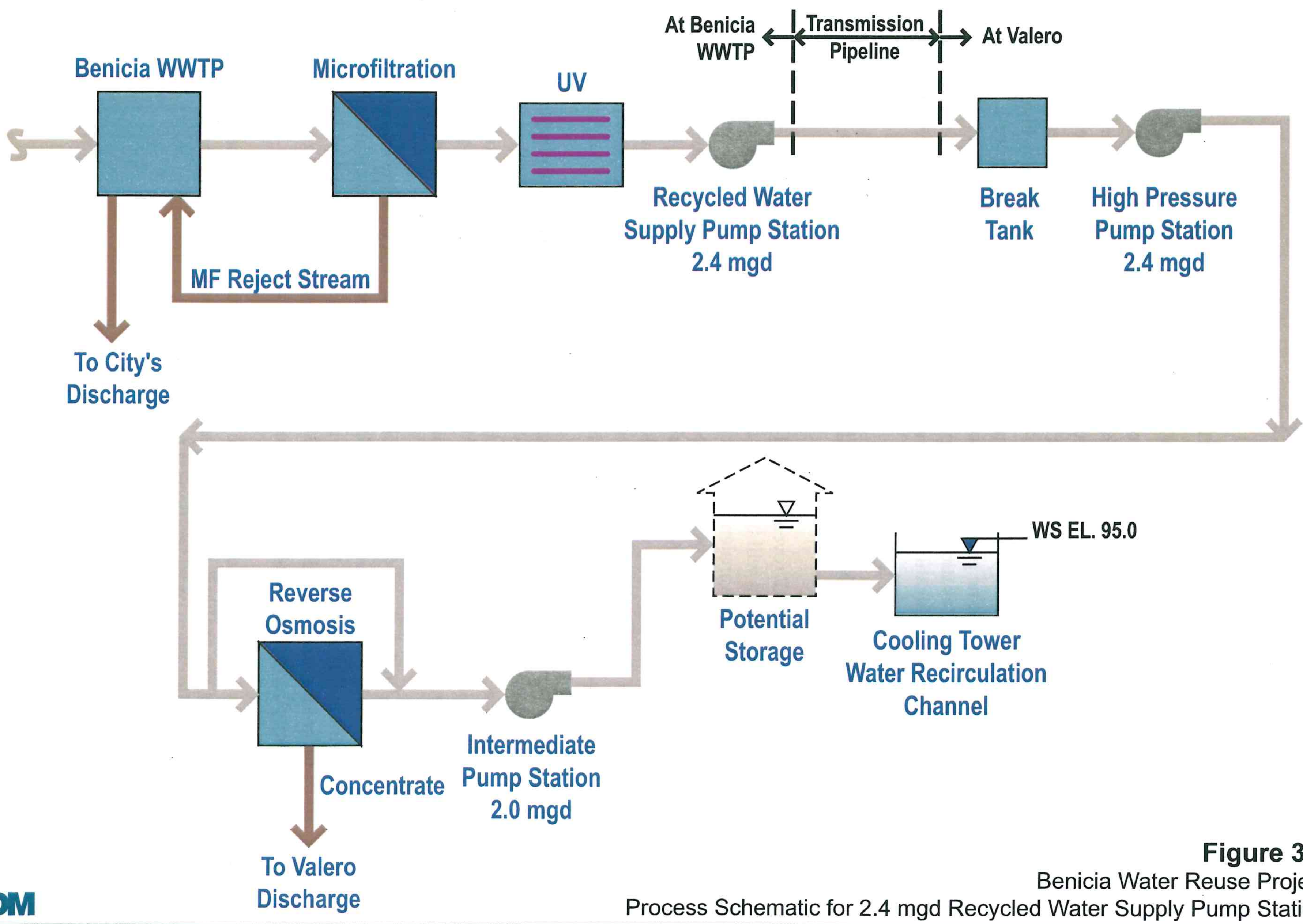


Figure 3.2
Benicia Water Reuse Project
Process Schematic for 2.4 mgd Recycled Water Supply Pump Station

CDM

4.0 Conveyance Pipeline Profile

4.1 Overview of Conveyance Pipeline Profile

Valero provided CDM with copies of plan and profile drawings of the "off-site" dock lines as well as information about the pipe material, pressure class, and wall thickness. Valero also provided information on the elevation of the existing pipeline sleepers within the Refinery. Using this information, CDM developed a preliminary profile of the pipeline from the City's WWTP to the cooling towers. Figure 4.1 contains a preliminary site plan alignment of the pipeline, and Figure 4.2 contains the preliminary pipeline profile. As shown in the profile, the high point is at approximately Elevation 201. The pump station at the WWTP will be located at approximately Elevation Zero. Hence, the static lift under either of the flow scenarios will be about 200 feet.

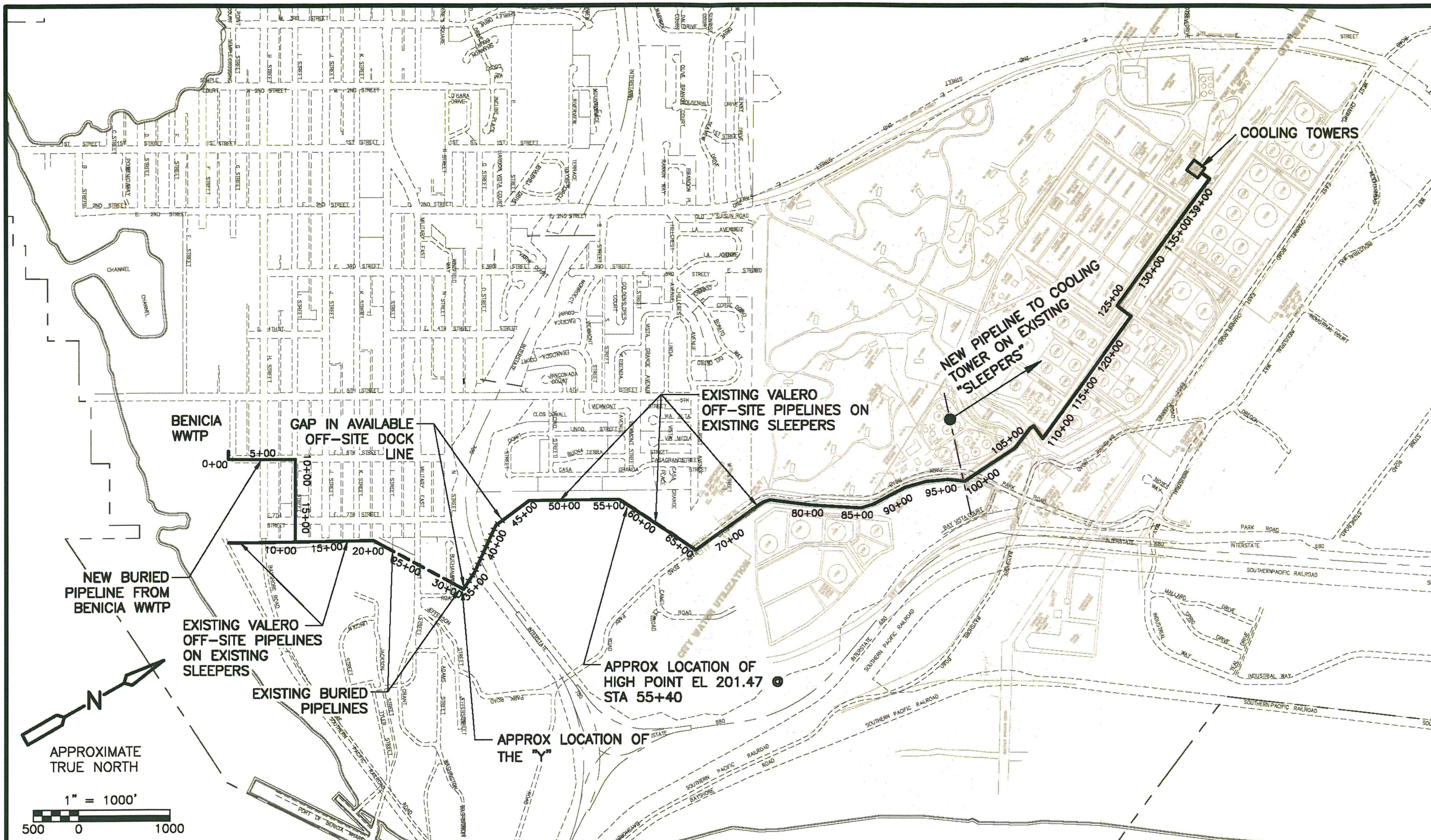
4.2 General Design Considerations of Conveyance Pipeline Profile

Review of Figure 4.2 reveals several intermediate high points, as well as sag points. The pipeline system will be subject to hydraulic transients if there is a sudden shutdown. These transients can cause excessive pressures to occur as well as vacuum condition, which must be taken into consideration in the design. Several combination air inlet and vacuum relief valves will need to be located at critical, high-point locations. A surge tank will potentially be required at the recycled water pump station at the WWTP. These facilities will control these pressures to within acceptable ranges that the pipe system can withstand.

At the low or sag points, blow-off valves will be required for draining the line and periodically "blowing down" solids that might accumulate in the line. If locations (sanitary sewer lines) to discharge the recycled water from these blow-off points are not available, then the water would have to be hauled off in tanker trucks. Discharge to storm drains would require the prior approval of regulatory agencies.

During final design, evaluations will be made of the best pipeline route from the WWTP to the connection on the Valero pipe sleepers. To avoid an intermediate high point in this segment, there may be an opportunity to install the pipe using a "no-dig" technique, known as directional drilling. The objective would be to avoid surface disruption and the cost of an air inlet and vacuum release valve.

Since over 90 percent of the pipeline will be installed above grade on existing or new sleepers, the profile is generally fixed. Therefore, adjustment of the profile to improve hydraulic design considerations is not an option.



CDM

THE CITY OF BENICIA
RECYCLED WATER CONVEYANCE SYSTEM

PIPELINE ROUTE MAP

FIGURE 4.1

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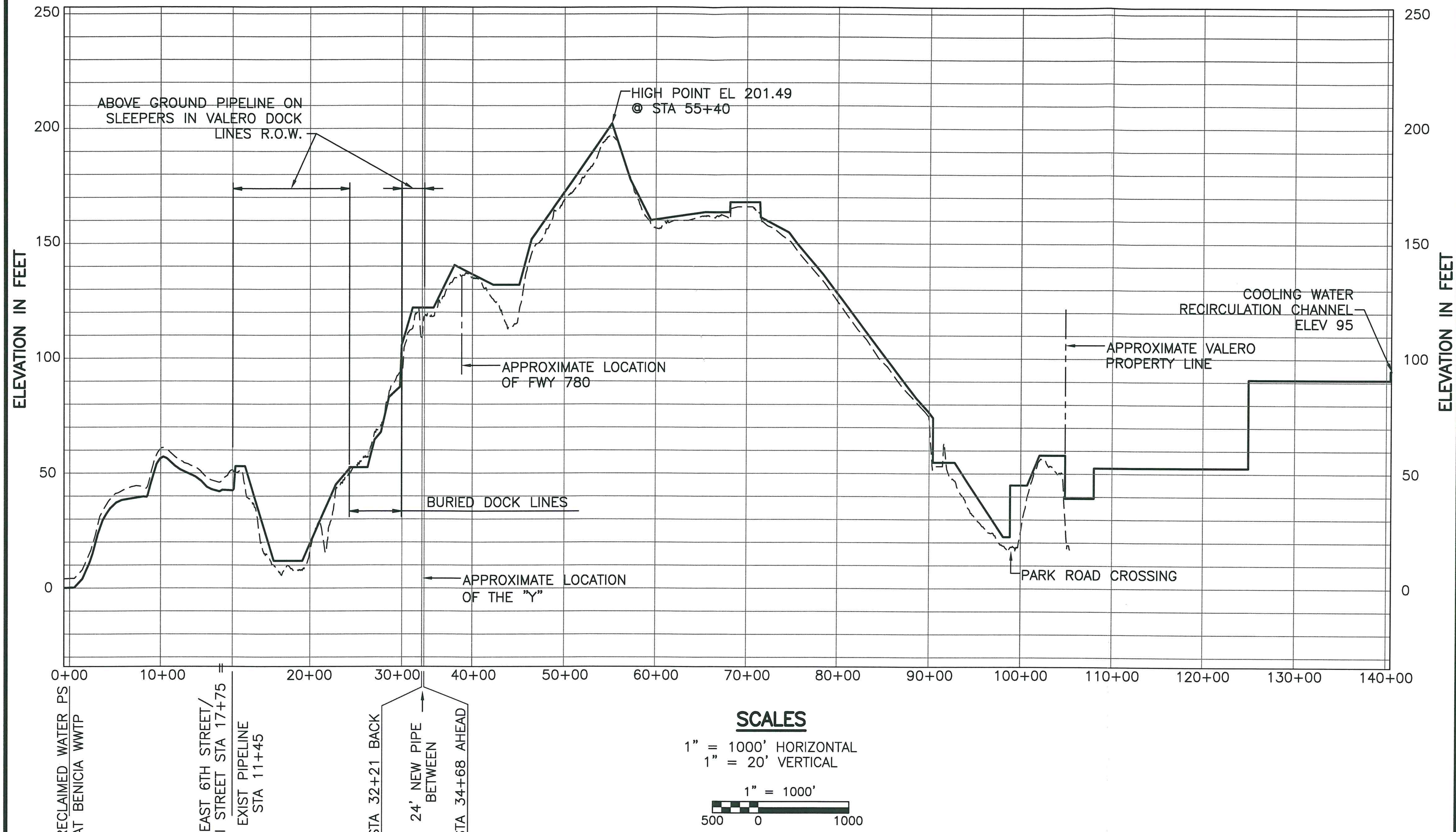

THE CITY OF BENICIA
RECYCLED WATER CONVEYANCE SYSTEM

PRELIMINARY PIPELINE PROFILE

FIGURE 4.2

CDM

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5.0 Description of Existing Valero Off-Site Pipelines

There are several pipelines along the off-site alignment, which runs from just east of the Benicia WWTP to the Valero Refinery property line, as shown in Figure 2.1. For a majority of this portion of the alignment, the pipelines are above-grade and are supported on pipe racks, known as "sleepers". The pipelines are below grade at two locations. The following table summarizes the physical characteristics of the pipelines, based on information provided by Valero. There are four or five "DL" or dock lines that are abandoned and potentially available for use on this project.

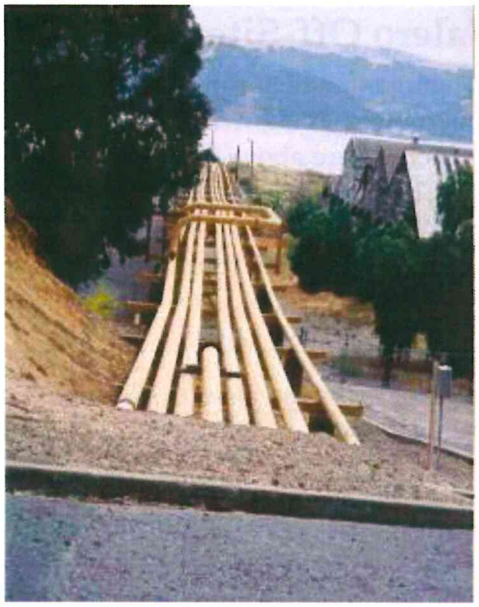
Table 5.1
Characteristics and Design Specifications of Existing Valero Dock Lines

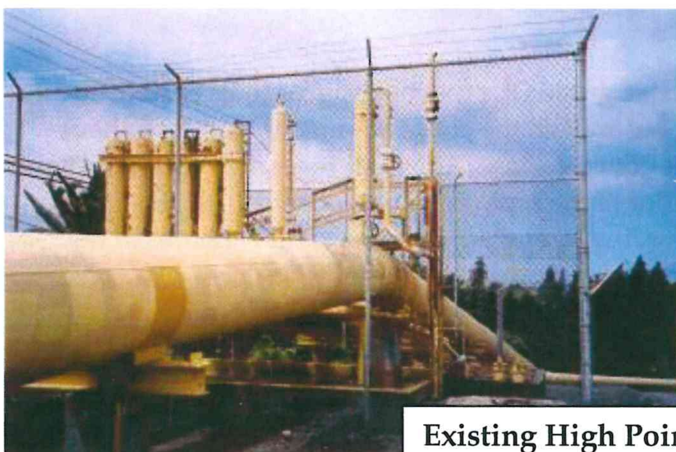
Line Name	Diameter, inches	Material	Operating Pressure, psig	Design Pressure, psig	Min Test Pressure, psig	Design Wall Thickness, inch
DL2	12	Steel	175	160	428	0.25
DL4	12	Steel	224	275	495	0.25
DL5	12	Steel	224	275	495	0.25
DL6	12	Steel	175	260	428	0.25

Based on field reconnaissance, the above grade DL pipelines appear to be coated, although CDM representatives have not walked the entire alignment. The pipes have not been inspected for exterior or interior corrosion. The steel pipes are continuously welded. Valero advises that the interior of the pipes is bare steel, which is fairly common for petroleum pipelines. Figures 5.1 and 5.2 contain example photos of the off-site and on-site pipelines, respectively.

5.2 Prior Pipeline Evaluations Performed by Valero

Valero has evaluated conveyance of recycled water through its existing pipelines since year 2000. Memos, prepared by or for Valero, recommended removing some of the existing dock lines not in use on the pipe sleepers, and replacing the lines with new HDPE piping, or cement-mortar lined steel pipes. (Appendix A contains copies of these memos.) The buried portion of the existing dock line was proposed to be used as a carrier pipe for slip-lining a new pipe. An internal Valero memo, dated February 2002, presented an estimate of constructing the pipeline, including pump stations at both the City's WWTP and within the Refinery, as well as a 2 million gallon storage tank. The estimated construction cost was stated at $7 million, including engineering and a 15 percent contingency. Another construction cost estimate was made in March 2002 by Underground Construction Co. Inc. That estimate was approximately $1.6 million and was based on pipe replacement with HDPE from the WWTP to the "Y" and cement-mortar lined steel pipe from the "Y" to the Refinery (assumed as the cooling towers within the Refinery), excluding pumping and storage. (The "Y" is shown on Figure 4.1 and is where the old DL's intersect the operating DL's.)





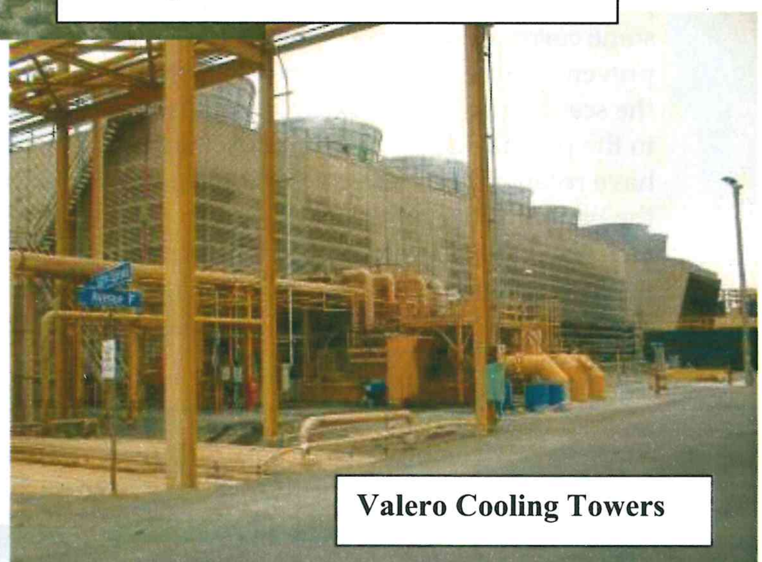
Existing High Point

Existing Pipelines on Sleepers just before Buried Section

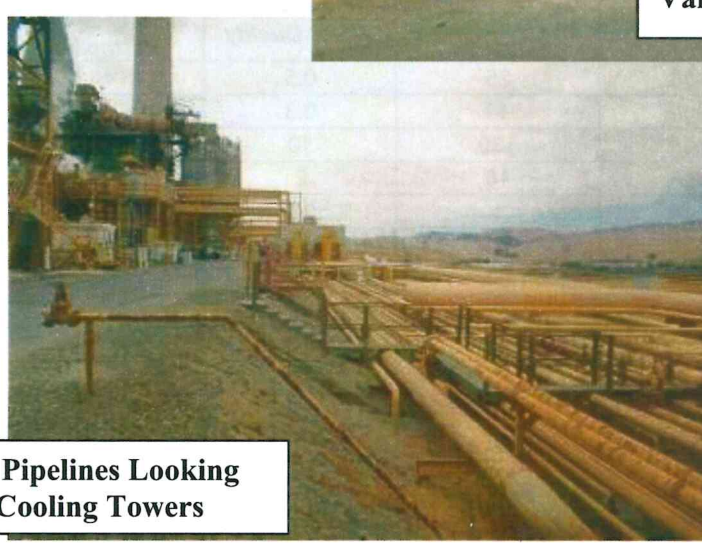
Existing Pipelines on Sleepers along Park Road

Figure 5.1
Example Photos of Existing, Valero Off-Site Pipelines

Cooling Water Recirculation Channel



Valero Cooling Towers



Valero On-Site Pipelines Looking North Toward Cooling Towers

Figure 5.2
Example On-Site Photos

6.0 Recycled Water Quality Considerations For Pipeline Design

To avoid corrosive environments within water transmission pipelines, chemical conditioning is frequently required. There are several corrosion indices, one being the Langelier Index, which is a measure of the tendency of a water to dissolve or deposit calcium carbonate. A positive index indicates a tendency toward deposition. One method to provide a positive index is to add a small amount of lime. The projected recycled water quality resulting from the proposed treatment system of MF followed by RO with 15 percent blending with MF filtrate was discussed in TM 1. The concentrations of several of the projected mineral characteristics of the recycled water are important for determining the types of potential rehabilitation lining systems as well as selecting the appropriate lining of new pipe. Table 6.1 presents the projected mineral quality of the recycled water. The low hardness may dictate that some corrosion control chemicals (i.e., "chemical conditioning") may be added to prevent it from becoming corrosive to the interior of the transmission pipeline. For the scenario in which the RO system is located at Valero, the recycled water conveyed in the pipeline may not require chemical conditioning since the recycled water will have received only micro-filtration; and from a mineral perspective, it will resemble the Benicia Effluent Water Quality, as shown in Table 6.1. However, it will be more of an issue if the pipeline conveys the blended water after the RO process, as noted by the low hardness, TDS, and bicarbonate. Alternative methods of chemical conditioning of the recycled water will be evaluated later in the preliminary design phase.

Table 6.1
Projected Blended RO Permeate, Recycled Water Quality

Parameter	*Units*	*Benicia Effluent Water Quality*	*RO Permeate Water Quality*	*Blended Water Quality @ 85% Permeate*	*Valero Cooling Water Quality Limits*
calcium	mg/L	25	0.5	4	
magnesium	mg/L	18	0.3	3	
sodium	mg/L	130	10	27	
potassium	mg/L	18	2	4	
ammonia	mg/L	1	0.3	0.4[1]	<0.2
bicarbonate	mg/L	190	11	37	104
sulfate	mg/L	90	1	14	
chloride	mg/L	120	4	21	20
phosphate	mg/L	2	0.2	0.5	3
fluoride	mg/L	1	0.1	0.2	
nitrate	mg/L	25	6	9	
silica	mg/L	22	0.7	4	17
hardness	mg/L	130	5	23	<200
TDS	mg/L	650	30	120	250

(1) Reduction of ammonia to < 0.2 mg/L will be accomplished by breakpoint chlorination.

7.0 Testing and Internal Inspection Methods For Existing Pipelines

7.1 Closed Circuit Television Inspection

Closed circuit television inspection services can be used to perform a visual inspection of the interior of the 12-inch diameter pipe before any lining is applied to determine existing conditions, and after lining is applied to inspect the finished lining workmanship. The camera is built on top of a remote controlled cart that is self-propelled or pulled through the pipeline to be inspected. The video images are transmitted back to an operator station inside a truck, where the camera and trolley are controlled. Access points are required at about every 1000 feet of pipe. The cost for video inspection is $2,600 per day for 1,800 feet of pipe, or about $1.44 per foot. The cost includes a two-person crew, explosion-proof camera and trolley, and video recording. This budgetary cost estimate is from Demakas Plumbing in San Francisco.

7.2 Pressure Testing

Pressure testing of the pipelines may be helpful in determining any locations of pipe deterioration or damage. Pressure testing is done using water as the test medium, pressurized at up to 150 percent of the expected operating pressure, but not above the design pressure of the pipe and fittings. A standard hydrostatic pressure test, as described in AWWA M11 Steel Pipe Design and Installation Manual, is acceptable. The test pressure should be maintained for at least two hours and there should be no significant leakage on an all-welded pipeline or one that has been joined with properly installed mechanical couplings. Pressure testing requires a temporary pump to pressurize the test section, pressure gages, and bulkheads. The approximate cost for testing is $2.50 per foot, and the entire line can be tested in 4 days. Repair clamps, about $1,000 each for materials and labor, will need to be used to repair pipe sections that have been cut to perform the pressure testing.

8.0 Description and Evaluation of Alternative Pipe Lining Systems

Although chemical conditioning can be used to inhibit the corrosivity of the recycled water, the Langelier Index is a measure of calcium carbonate which is of concern with cement mortar lining. However, there are other potentially corrosive aspects of the recycled water that could attack bare steel, by setting up galvanic corrosion at "weak" sites on the interior pipe surface. Hence, it is appropriate to line the bare steel to avoid this potential situation.

Six types of lining systems were reviewed to determine their applicability to the existing Valero Dock Lines and to obtain conceptual unit costs. Each system is described below.

8.1 Fold and Form Liner

Fold and form liners are most commonly used on gravity and low pressure systems and are generally made of PVC or polyethylene. For installation, they are temporarily folded or formed to reduce their size before insertion into the host pipe. After insertion, they revert to their original profile to create a close fit liner by using water pressure and then they are heated so that they cure onto the host wall material. Photo 1 shows the folded liner on the left, and a cured liner on the right. Photo 2 shows a folded liner being inserted into a sewer line. Fold and form liners are manufactured by many companies including AM-Liner, SWPipe, and Dupont. ASTM F1867 and F1871 standards describe the materials and requirements for installation of this type of liner system.

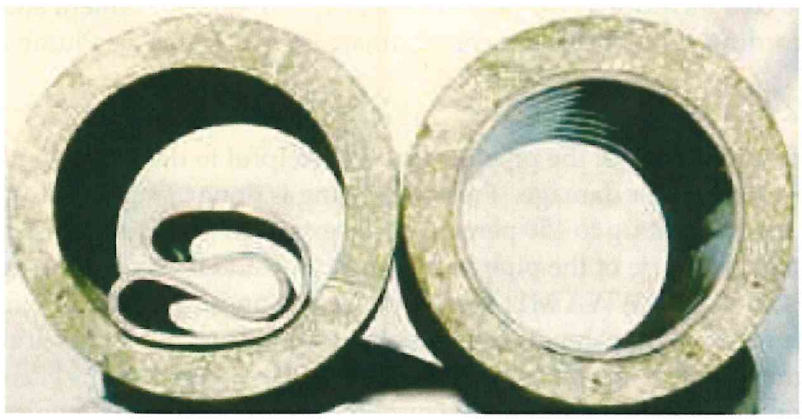


Photo 1:
Shows folded and cured liner. *Courtesy of AM-Liner.*

Photo 2:
Shows folded liner being inserted into existing pipe.
Courtesy of AM-Liner.

8.2 Cured-In-Place Pipe (CIPP)

CIPP is a widely used method for rehabilitating pipes. CIPP is a lining system that uses a fabric tube impregnated with a thermosetting resin. The tube is pulled into the host line, inflated against the pipe wall, and then cured. The fabric tube is tailored in a factory to suit the diameter of the host pipe. The curing of the resin can be achieved chemically, thermally, or by ultraviolet light. Hot water is most commonly used to inflate the tube and is maintained in the tube until the resin cures against the host pipe wall. CDM's experience with CIPP has found that it is not well suited for lining pipelines that will experience operating pressures above 50 psi. CIPP is manufactured by many manufacturers including InSitu-Form, CIPP-USA, and SanCon. ASTM F2019 and D5813 describe the materials and installation of CIPP.

8.3 Duraliner

An alternative type of liner similar to CIPP but made of PVC material that can withstand up to 150 psig with a 2.3 safety factor is a product called Duraliner, that is manufactured by Underground Solutions. CDM has limited, but overall positive, experience with this product. The liner is made of fusible PVC pipe that meets AWWA C905 and NSF 61 standards. Twenty-foot lengths of stock PVC pipes with smaller diameter than the host pipe are field-welded and inserted into the larger host pipe. Heat or pressure is used to expand the liner pipe. After cooling the PVC liner stays in place, but does not adhere to the host pipe. The Duraliner can only be used on straight runs of pipes and cannot accommodate any bends and turns. The manufacturer recommended replacing bends and turns with ductile iron fittings. The host pipe needs to be cleaned to remove debris prior to installing the liner. The cost of Duraliner is in the range of $55 to $70 per linear foot installed. The oldest Duraliner installation is 3 years old in Louisville, Kentucky, on a 6- to 12-inch diameter water system.

8.4 Swage Lining

In this method, the diameter of the polymeric liner pipe is temporarily reduced by passing it through a set of dies, a process known as swaging, or through rollers. The reduced liner pipe can then be inserted into the host pipe using conventional sliplining techniques, in which the liner pipe is pulled through the carrier pipe by a cable attached to the liner pipe. The liner retains in its memory the original larger diameter to which it will revert. In the simplest form, reversion will be achieved when the tension on the winch rope is released, although pressurizing the liner may help. Pipe reduction takes place on site, and the tension on the liner must be maintained until it is in position. Once tension on the drawn line is removed, the liner starts to revert to its original diameter to fit snugly against the host pipe wall. Common materials used in Swage lining include polyethylene and polypropylene. Swage lining has been used extensively in the rehabilitation of gas and petroleum lines in Europe but has not been used much in the United States. Swage lining is manufactured by Insituform (United Titeliner) and Advantica (Swage lining). Swage lining costs $50 to $60 per linear foot installed.

8.5 Epoxy Lining

Liquid epoxy consists of primer and one or more coats of chemically cured epoxy coat finish. The epoxy coating has high flexibility, elongation, and impact resistance. Epoxy lining is applied in 10 to 20 mil. Epoxy spray lining can accommodate most bends, change in pipe diameters, and pipe irregularities. It requires minimal removal of fittings. Epoxy lining is not meant to provide or add structural strength to the pipe, so the pipe should be intact for this type of lining. AWWA C210 describes liquid epoxy coating and lining systems for water service.

Prior to applying epoxy lining, the interior of the pipe to be lined first has to be sand blasted to create a surface profile for the lining to adhere. Although there is more than one method of applying epoxy lining, the most controlled method is by using an application nozzle head that sprays epoxy radially out onto the interior surface of the pipe. The nozzle head is pulled through the pipe at a controlled rate to obtain uniform coverage. The nozzle is shown in Photo 3.

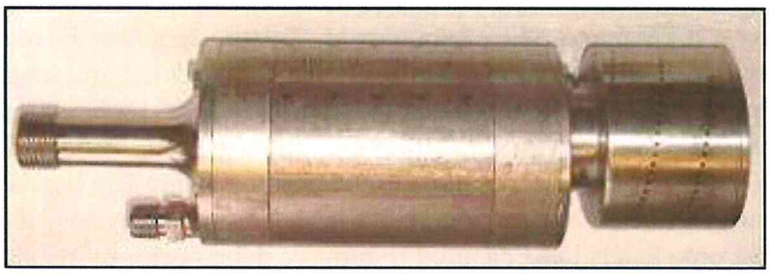


Photo 3.
Epoxy Lining Spray Nozzle
(source: J&F Tools, Ltd)

American Pipelining of San Diego provided the price estimate for linear foot of epoxy lining including sandblasting preparation. The estimated budgetary cost is approximately $65 per linear foot.

8.6 In-Situ Cement Mortar Lining

Cement mortar is composed of Portland cement, sand, and water, mixed together to form a homogenous lining material. The cement mortar is centrifugally spun onto the interior of the pipe for a smooth, uniform surface. AWWA C205 describes the material and application requirements to provide protective linings for steel water pipe by shop application of cement mortar. Cement mortar is field applied in a similar way to epoxy lining. The estimated unit cost of in-situ cement mortar lining is $100 per foot.

8.7 Summary of Pipe Lining Systems

Table 8.1 summarizes the pipe lining alternatives. For all of the options, the budgetary unit prices do not include the costs for cutting pipes for access and reassembling pipes. The average installation length for any liner is about 500 feet (because of pipe bends), so there would be about 15 pipe access locations required. Pipe access could be anywhere along the welded pipe or at elbow flanges or joints. To rejoin the pipes, pipe repair clamps will need to be used. The cost to cut and repair the pipes is estimated to be $2,000 each.

Based on the discussions above and the summary presented in Table 8.1, Duraliner and Swage lining are the preferred alternatives because they are able to withstand the reclaimed water system pressure, do not require extensive interior surface preparation and can accommodate abrupt changes in alignment. Both of these lining methods have similar estimated unit costs per foot, as well as similar access point requirements. The total unit construction cost of lining was estimated at $80 per lineal foot of pipe.

Table 8.1
Summary of Pipe Lining Alternative

Type of Lining	*Liner Material*	*Length of Installation between Access*	*Intended Service*	*Approximate $ per Foot (incl Labor)*	*Experience in USA*	*Other limitations*	*"R" Recommended "NR" Not Recommended*
Fold and Form	PE or PVC	500 ft, can do about 500 ft per day	Gravity or low pressure	$40 to $50	Good	Can negotiate some bends, but limited. Requires heated curing.	NR Can't meet pressure requirement
Cured-In-Place Pipe (CIPP)	PE	700 ft, can do about 700 ft per day	Gravity or low pressure	$65	Good	Can negotiate some bends, also needs a tower of water for inverting the pipe. Requires heated curing.	NR Can't meet pressure requirement
Duraliner™	PVC	500 ft, can do about 500 ft per day	Pressure 100-200 psi	$55 to $70	Minimum	For straight runs of pipes .	R
Swage	PE or PP	Can do about 2500 ft of straight pipe/day, limited by access	Pressure 100-200 psi	$50 to $60 (includes surface prep, CCTV)	Minimum, more in UK for petroleum industries	Not for deformed or irregular pipes.	R
Epoxy	Epoxy or Polyurethane	400 ft segment, can do about 400 ft per day of coating	Pressure 100-200 psi	$65 (includes surface prep)	Mostly done on 6- 10" dia. pipes in high rise buildings.	Requires sand blasting preparation of pipe for coating adhesion. Special equipment required. May require heat for curing.	NR Difficult surface preparation
Mortar	In-Situ Cement Mortar		Gravity or low pressure	$100	Minimum for 12" dia pipe size	May require careful water conditioning. Needs good surface preparation.	NR Difficult surface preparation

9.0 Alternative Materials for New Pipe

9.1 Pipe Materials for Buried Segments

Appropriate pipe materials to be considered for the buried segments include:

- HDPE
- Cement mortar or epoxy lined and mortar coated steel
- Cement mortar or epoxy lined ductile iron

The selection of which materials to use will be narrowed during the design phase. Alternative pipe materials will be allowed in the bid and construction phase.

Pipe Materials for Segments on Sleepers

Steel pipe is the best choice for pipe sections installed on sleepers. HDPE has a large coefficient of thermal expansion, which would require special design and construction features. Also, it lacks the strength to span the 30-feet on sleepers without excessive deflection. Hence, intermediate supports would be required. Also, since HDPE is flammable, it is not allowed within the Refinery. For interior corrosion prevention measures, CDM recommends that the steel pipe be cement mortar lined.

Selecting the type of joints for the steel pipe requires careful consideration. Because the pipe will be placed on sleepers, it must be capable of structurally supporting itself between sleepers, when full of water, and at the same time resist seismic forces. Hence, push-on gasket joints are not applicable and mechanical joints (restrained or grooved) will not accommodate bending stresses. Therefore, flanged or welded joints can be used. The difficulties with flanged joints are that they are very expensive and not accommodating for variations that will be encountered for actual field conditions.

10.0 Alternatives to Rehabilitate or Replace Existing Valero Pipelines In Off-Site Right-Of-Way

10.1 Description of Alternatives

There are basically two alternatives for the segment of the pipeline from the lower end of the dock lines (Sta 11+45) to a location approximately 2,000 feet south of the Refinery property line and entrance road (Sta 85+20). Those two alternatives are: (1) Constructing a new 14-inch diameter pipe, mounted on the existing sleepers; and (2) Rehabilitating the existing dock line segments that are available in the reach and providing new 14-inch diameter pipe in locations where there are gaps. For all new pipe, 14-inch diameter has been selected based on an analysis of the present worth of the difference in power costs between 12- and 14-inch diameter pipe showed that 14-inch pipe is substantially more economical. The difference in the 20-year present worth of the power cost was estimated at approximately $250,000 for a flow of 2.0

mgd and $350,000 for a flow of 2.4 mgd (for the scenario of the RO located at the refinery). At a basic material cost difference of approximately $5 per foot, the cost of the larger pipe clearly offsets the increased cost of pumping.

Hence, two alternatives were evaluated, as follows:

- Alternative No. 1 – New pipe at 14-inch diameter
- Alternative No. 2 – Rehabilitate pipe plus new 14-inch diameter pipe

10.2 Bases of Pipeline Cost Estimates

10.2.1 Construction Cost Estimates

- Pipe Rehabilitation Costs – Pipe rehabilitation cost estimates were based on the unit cost of lining, using either Duraliner or Swage lining as presented in Section 7 above and the estimated cost of thorough testing as presented in Section 8, above. A consolidated unit cost of $80 per foot of rehabilitated pipe was used.
- New Buried Pipeline Costs – Estimated unit costs for buried pipeline was developed from experience on similar recent projects, including the City's wet weather interceptor, and were taken into consideration in determining the estimated unit cost of $125 per foot.
- New Pipeline Mounted on Existing, Off-Site Sleepers – Estimating costs for 14-inch cement mortar lined steel pipe was obtained from pipe suppliers. Production rate was estimated at 300 feet per day with a crew of five plus a crane and operator. An estimated unit cost of $75 per foot was used.
- New Pipeline Mounted on Existing Sleepers Within the Refinery – In addition to the unit cost of $75 per foot developed for new pipe on existing, off-site sleepers, an additional $10 per foot was added for framing extensions that would need to be added to the existing sleepers because they are full of pipes for nearly the entire in-plant alignment.
- Valves – Estimated costs for air release and vacuum valves, in-line valves and blow-down valves were taken from experience on recent projects and include an allowance for the associated fittings, required to incorporate them into the piping system.

Contractor's overhead and profit are included at 15 percent. Owing to the level of detail developed in this conceptual design phase, a contingency allowance of 25 percent is included to account for lack of detailed information, estimating variances, and relatively small items that may not have been included.

10.2.2 Operating and Maintenance Cost Estimates

Electrical power costs used are $0.12/kWhr, which is based on the average unit price for power at the WWTP for one winter month and one summer month. O&M labor is assumed the same for either alternative.

10.3 Economic Evaluation of Alternatives

The capital and annual O&M cost estimates presented herein are for comparative purposes only. These cost estimates are used to determine to develop life cycled costs and then to determine if it is more cost effective to rehabilitate the existing dock line or to replace it with a new pipeline. The evaluation includes only the pipe segment from Station 11+45 (at the lower end of the pipeline near the intersection of "I" Street and 7th Street) to Station 85+20 (which is the end point of the existing 12-inch pipe, potentially available for rehabilitation). However, the analysis does not include the buried portion from approximately Station 24+60 to Station 30+00, because this section will need to be lined and rehabilitated under either alternative (for a length of approximately 540 feet). The analysis also includes the increased energy cost for the portion of the alignment above, as well as increases in motor sizes (if any) of the recycled water pumps. Also, a flow of 2.0 mgd has been assumed in the analysis, because whichever alternative is more cost-effective at the lower flow rate would also be more cost-effective at the higher flow rate, owing to impacts of power cost on the analysis. Lastly, it has been assumed that whatever special valves (air release, blow down, etc.) are required would be common costs for both alternatives.

As discussed above, approximately 6,610 feet of new pipe would be required under Alternative No. 1, New Pipe. Under Alternative No. 2, Rehabilitate Pipe, approximately 770 feet of new 14-inch pipe would be required and approximately 5,840 feet of existing 12-inch diameter pipe would require rehabilitation. A review of Table 10.1 shows that replacing the pipeline in the reach between Station 11+45 and Station 85+20 is approximately 18 percent more economical than rehabilitating the existing pipe segments within this 6,610-foot reach.

10.4 Qualitative Evaluation of Alternatives

Risk of failure and unknown pipeline conditions are major factors to be considered in selecting the rehabilitation alternative. Also, although not readily quantifiable, pipeline maintenance may be higher with the rehabilitated system. As noted in Summary Table 7.1, most lining systems do not have a long, in-place history; hence, their longevity is not well established. Lastly, once rehabilitation construction is underway, changed conditions would likely occur that would lead to increased costs.

10.5 Recommended Pipeline Alternative

Given the uncertainties associated with rehabilitation, CDM recommends that the conveyance pipeline be constructed completely of new 14-inch diameter pipe. Pipe materials will be evaluated further during preliminary and final design.

Table 10.1
Summary of Economic Analysis of Alternative Pipeline Systems for Off-Site Piping

	Unit Costs	Alternative No. 1 New Pipe	Alternative No. 2 Rehab Pipe
Length of 12-in Rehabbed Pipe, ft	n.a.	0	5,840
Length of New 14-in Pipe, ft	n.a.	6,610	770
Unit cost of New 14-in Pipe, $/ft	$75	$496,000	$58,000
Unit cost to Rehab 12-in Pipe, $/ft	$80		$467,000
Subtotal Construction Cost		$496,000	$525,000
Contingency, 25%		$124,000	$130,000
Subtotal		$620,000	$655,000
Contractor OH & Profit, 15%		90,000	100,000
Total Estimated Construction Cost		$710,000	$755,000
Add 35% for Engr & CM		$250,000	$265,000
Total Estimated Capital Cost		$960,000	$1,020,000
Power Required, kWhr/yr		67,000	158,000
Assumed Power Cost, $/kWhr	$0.12	$8,000/yr	$19,000/yr
Present Worth Power (i=6%, t=20yr)		$92,000	$218,000
Total Estimated Present Worth[1]		**$1,050,000**	**$1,240,000**

[1] Rounded to the closest $10,000

11.0 Conceptual Design of Recycled Water Conveyance Pipeline

11.1 Description and Estimated Construction Cost of Total Recycled Water Conveyance Pipeline

Based on the analysis presented above in Section 9, the recycled water conveyance pipeline will be an entirely new pipeline, 14-inches in diameter and mounted on existing sleepers for the majority of the alignment. The preliminary profile is shown in Figure 4.2. There is one exception to "an entirely new pipeline" and that is between Stations 24+60 and 30+00, where two of the existing 12-inch pipelines that are buried will be rehabilitated by lining them to avoid surface construction in the segment. Table 11.1 presents the estimated construction cost for the main conveyance pipeline from the Benicia WWTP to the Valero Cooling Towers. As shown in Table 11.1, the estimated construction cost of the main conveyance pipeline is $2.06 million. As noted on the table, this cost estimate does not include the costs to analyze and rehabilitate the existing sleepers.

Table 11.1 ***Estimated Construction Cost of Total Recycled Water Conveyance Pipeline***			
Component	***Unit Cost***	***Quantity***	***Estimated Costs, $1000's***
Segment No. 1: Sta 0+0 @ Benicia WWTP to Sta 17+75 @ connection to sleepers. Construct new, buried 14-in pipeline	$125/ft	1,775 ft	$222
Segment No. 2: Sta 11+45 @ start of sleepers to Sta 24+60 @ start of existing, buried 12-in lines. Construct new, 14-in pipe on existing sleepers	$75/ft	1,315 ft	$99
Remove Dock Line No. 3 from sleepers in Segment No. 2	<$5>	1,315 ft	<$7>
Segment No. 3: Sta 24+60 to Sta 30+00 end of existing, buried 12-in lines. Rehabilitate and connect to 2, existing 12-in lines	$250/ft	540 ft	$135
Segment No. 4: Sta 30+00 to Sta 32+20 at the "Y" plus additional 30 ft. Construct new, 14-in pipe on existing sleepers	$75/ft	250 ft	$19
Remove Dock Line No. 3 from sleepers in Segment No. 4	<$5>	220 ft	<$1>
Segment No. 5: Sta 34+68 at the "Y" to Sta 42+15, end of where existing 12-in DL has been removed. Construct new, 14-in pipe on existing sleepers	$75/ft	747 ft	$56
Segment No. 6: Sta 42+15 to Sta 85+20, end of existing, abandoned 12-in DL. Construct new, 14-in pipe on existing sleepers	$75/ft	4,305 ft	$323
Remove abandoned pipe from sleepers in Segment No. 4	<$5>	4,305 ft	<$22>
Segment No. 7: Sta 85+20 to Sta 105+00, approximate Valero PL. Construct new, 14-in pipe on existing sleepers.	$75/ft	1,980 ft	$149
Segment No. 8: Sta 105+00 to Sta 140+00, approximate location of cooling towers. Construction new, 14-in pipe on extensions to existing sleepers.	$85	3,500 ft	$298
6-inch Air Inlet and Vacuum Release Valves	$10,000 ea	4	$40
2-inch Air Inlet and Vacuum Release Valves	$7,000 ea	5	$35
6-inch Blow Down Valves (BV's)	$4,000 ea	6	$24
14-in In-Line Isolation Valves (BV's)	$8,000 ea	7	$56
Subtotal[(a)]			$1,430
Contingency at 25%			$360
Subtotal			$1,790
Contractor OH and Profit at 15%[(1)]			$270
Total Estimated Construction Cost[(2)]			**$2,060**

(1) Rounded to closest $10,000's.

(2) Cost estimate does not include the costs to analyze and rehabilitate the existing sleepers.

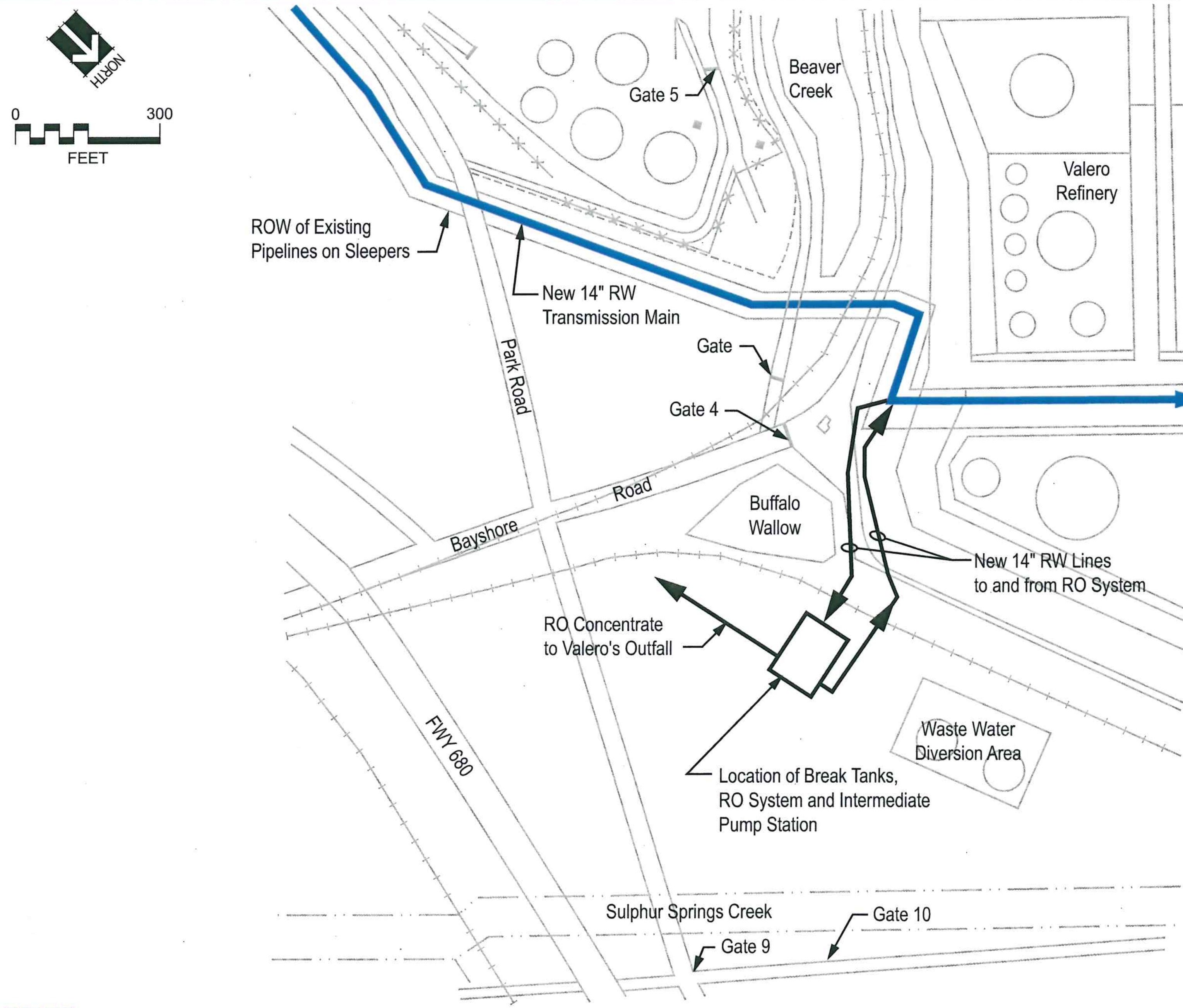
11.1.1 Additional Pipelines Required at Valero for 2.4 mgd Scenario

As discussed in Section 3 above, there are two potential design flow scenarios depending on where the RO treatment system is located, either at the City WWTP or at the Valero Refinery. The design of the main transmission pipeline is basically the same for either flow scenario; however, the pipeline components at the Refinery are different for each scenario. If RO is located at the City's WWTP, then the pipelines will be constructed directly to the cooling tower area. However, if the RO is located at the Refinery, pipelines leading to the location of the RO system and then conveying the permeate from the RO system to the back to the main transmission pipeline would need be included in the Project. Figure 11.1 shows a conceptual plan of these two scenarios in the area near Gate No. 4 to the Refinery off Bayshore Road. As shown in the figure, the RO system, including break tank, high pressure pump station (to feed the RO system) and intermediate pump station to lift the recycled water back up to the sleepers, would be located near the Refinery Waste Water Diversion Area. (This is the location that was recommended by Valero in a phone conversation between Steve Penny of Valero and Jerry Cole of CDM.) Ground elevation in this area is approximately zero. A connection would be made at approximately Station 107, where the main pipeline jogs northerly, as shown on Figure 11.1. A return line from the new, intermediate pump station would also be required. Neither the costs of these pipelines nor the intermediate pump station have not been included in the estimate. The cost of these facilities will be taken into account in evaluating the economics of where to locate the RO Facilities, which is the subject of future TM-4, Siting of Facilities.

12.0 Conceptual Design of Recycled Water Supply Pump Station

12.1 Physical Description

The Recycled Water Supply Pump Station (RWSPS) will receive flow from the Water Reuse Treatment Plant (WRTP). It will be a component of the WRTP and the proposed location is adjacent to the WRTP UV channels. A cast-in-place concrete structure with the lower level being a clearwell is recommended. The pumps will be vertical turbine type, mounted outdoors on a concrete deck over the clear well. Electrical equipment for the pumps will be housed in the electrical room of the control building for the WRTP. Recycled water will flow from the UV channels into the RWSPS clearwell from where it will be pumped into the recycled water conveyance system by the RWSPS pumps. The pump motors, discharge piping and valves, and monitoring and sampling equipment will be located in the deck area over the clearwell. The pump station will be rectangular in shape with the plan dimensions being determined based on pump and other equipment space requirements in the pump deck area and to a secondary extent, storage volume in the clearwell.



CDM

Figure 11.1
Site Plan for Alternative Location for RO System

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The design of RWSPS will allow for future expansion to increase supply to Valero or to supply other future recycled water users.

12.2 Pump Selection

Preliminary pump selection is based on the preliminary pipeline profile (refer to Figure 4.2) and the recommendation that a new, 14-inch diameter pipeline be constructed for the transmission main. A preliminary hydraulic analysis was performed based on the two flow criteria, namely 2.0 mgd and 2.4 mgd. Pump equipment vendors were contacted to determine the preliminary economics of providing one or two duty pumps. Based on the estimates provided, although it would cost less to provide only one duty pump, it was also determined that, owing to the high static head, one duty pump could not be turned down to 50 percent output capacity and still remain on the system curve. Also, because there will be two RO banks with 50 percent capacity, it makes sense to provide pumps that can easily meet one half the system design capacity. CDM's preliminary recommendation is that all pumps will be variable speed to provide for variations in demand. The number of pumps and variable speed versus constant speed will be evaluated further in the preliminary design phase.

The plant hydraulic profile will establish the maximum water level in the RWSPS clearwell. The bottom elevation of the clearwell will be established based on the required operating level differential relative to the high level, minimum pump submergence and vertical distance between the clearwell and the pump intake. The clearwell will be provided with concrete baffle walls in accordance with Hydraulic Institute Standards to improve pump performance and avoid vortexing.

12.3 Mechanical Design Considerations

12.3.1 Surge Control

A surge analysis of the entire conveyance system, including identification of alternative mitigation measures, will be performed during final design. Preliminary indications are that a surge tank will be required at the RWSPS.

12.3.2 Valves and Appurtenances

Each pump discharge will have a manual isolation butterfly valve and a check valve. Due to the high discharge head and potential surge conditions, it is anticipated that the check valve will be the double-door, fast-acting, silent type.

A manual isolation butterfly valve will also be provided on the discharge header downstream of the flow meter to isolate the meter from the transmission line.

Each pump discharge will also have an air release valve to release air on pump start-up. The air release valve will be specially designed for use on vertical turbine pumps and will contain an air pressure release throttling mechanism. Air release valves will also be provided on the pump discharge header at high points where air may accumulate.

12.4 Electrical Design Considerations

Power supply to the motors will be 480-Volt, 3-phase, 60-Hz power fed from a MCC located in the new MF/RO equipment building.

12.5 Instrumentation, Monitoring and Control Design Considerations

12.5.1 Pump Control

The RWPS pumps will be automatically controlled by the WRTP PLC based on water level in the clearwell. In that way, RWPS will match the production rates of the WRTP, which will be controlled to match average daily demand. The pumps will also be able to be controlled to pump at a selected flow rate by setting a specific rate through the PLC. Manual pump start and stop and speed control will also be provided at the WRTP PLC.

Control interlocks with other systems will be as follows:

- All of the RWSPS pumps will be automatically stopped on high level in the break tank (or storage tank) at Valero to avoid overfilling the tank.
- All of the RWSPS pumps will be automatically stopped on high micro-filtration effluent turbidity conditions.
- All of the RWSPS pumps will be automatically stopped on detection of critical alarm conditions at any of the upstream treatment processes.
- Under any of the hydraulic or process performance alarm conditions that would shut down the pumps, the recycled water would be routed to the City's outfall until the alarm conditions have been addressed and cleared.

12.5.2 Monitoring

The following are monitoring provisions:

- Water level in the clearwell will be continuously monitored using an ultrasonic level sensor, with separate float switches for high and low level alarms in the event of failure of the level sensor. The water level signal will be used for pump control as described above.
- A magnetic flow meter will be provided on the pump discharge header to measure pump flow rate. The flow signal will be used for regulatory and recycled water inventory recordkeeping, for RWPS monitoring and for pump control as described above.

- A pressure transducer will be provided on the recycled water discharge header to continuously measure header pressure for the purposes of monitoring pump operation and head conditions in the transmission system.

- A locally indicating pressure gauge will be provided on the discharge header and on each pump discharge.

12.5.3 Equipment Protection

The following equipment protection measures will be considered in the design phase:

- Monitoring of motor winding and bearing temperature with automatic pump shutdown on high temperature condition.

- Due to the relatively high operating pressures, providing pumps, vibration monitoring with automatic pump shutdown on high vibration condition.

- A discharge flow switch for each pump or check valve position monitor to confirm that flow is actually occurring. No flow would generally indicate that the pump is spinning at near shut-off head.

12.5.4 Sampling

A refrigerated autonomic composite sampler may be required for regulatory sampling. The RWQCB may require the City to sample and report the quality of recycled water leaving the City's property. The sampler would draw from the recycled water discharge header and would be flow paced from the RWSPS flow meter.

12.6 Design Criteria

Preliminary design criteria for the RWSPS are presented in Table 12.1.

12.7 Estimated Construction Costs

The estimated construction costs for the two sizes of pump stations pump were based on the following assumptions, unit costs, and budgetary quotes from vendors:

- <u>Foundations</u> – Owing to the poor soil conditions (Bay mud) in the area available for the project, it will be necessary to place new structures on pile foundation systems. Based on review of the Geotechnical Engineering and Environmental Services Report, dated 15 July 1997 and prepared by Harza Engineers for the City's 1998 WWTP Improvement Project, pre-cast concrete piles, driven to an approximate depth of 70 feet, have been assumed. Conceptual design estimates were made of the number of piles per structure, plus mobilization and demobilization. Pile driving costs were assumed at $40 per foot of pile, including the cost of the pile. Estimates were based on budget quotations obtained from a local pile driving subcontractor.

Table 12.1
Recycled Water Pump Station Design Criteria

Criteria	*Units*	*2.0 mgd System*	*2.4 mgd System*
System Pumping Requirements			
Design Capacity	Mgd	2.0	2.4
Design Capacity	Gpm	1,400	1,680
Design TDH	Ft	250	265
Static Head	Ft	200	200
Pump Units			
Type		Vertical Turbine	
Number, Total/Duty/Standby		3/2/1	3/2/1
Design Capacity per Pump		700	840
Design TDH per Pump	Ft	255	270
Min. Efficiency at Design Point	%	82	
Stages per Pump	No.	4	4
Pump Operation		Variable	
Minimum Speed	Rpm	TBD	TBD
Pump Motors			
Type		TEFC	
Size, each unit	Hp	60	75
Drive Type		VFD	
Synchronous Speed	Rpm	1,800	
Power Supply		480-V/3-phase/60Hz	
Pump Discharge Piping			
Diameter	Inch	8	8
Velocity at Design Flow	Fps	4.43	5.32
Pumps Discharge Header Piping			
Diameter	Inch	14	14
Velocity at Design Flow	Fps	2.90	3.48
Discharge Flow Metering			
Type		Magnetic	
Size	Inch	10	10
Velocity at Design Flow Rate	Fps	5.67	6.81

- Structural – Pump station wet wells were assumed to be constructed of cast-in-place reinforced concrete and were estimated at $1.5 per gallon capacity. Structural costs include excavation, reinforced concrete, and structural backfill.

- Civil – Civil site work costs were estimated at 20 percent of structural costs (excluding foundation costs) to cover site preparation, grading, paving, and site piping.

- Mechanical – Mechanical equipment costs were obtained from vendors and/or were based on experience from other similar projects. Budgetary costs for pumps and motors were obtained from J.M. Squared Associates and for VFD's from Robicon.

- Electrical – Power supply will be required for the pump motors and related components. Electrical costs were estimated at 30 percent of the mechanical equipment cost based on experience with construction of similar systems.

- Instrumentation - Instrumentation will be required for process monitoring and control and for connection to the plant SCADA system. Typical instrumentation includes monitoring of flow rate, wet well water level, pump operating conditions, and others. The instrumentation costs are estimated at 20 percent of mechanical equipment cost.

Contractor's overhead and profit are included at 15 percent. Owing to the level of detail developed in this conceptual design phase, a contingency allowance of 25 percent is included to account for lack of detailed information, estimating inaccuracies, and relatively small items that may not have been included.

Based on the design criteria and bases of cost estimates, presented herein, conceptual cost estimates were developed for two RWSPS's for the two flow scenarios discussed above. The cost estimates are contained in Table 12.2.

Table 12.2
Estimated Construction Costs – Alternative Recycled Water Supply Pump Stations

Items	***Quantities***	***Unit Prices ($/unit)***	***2.0 mgd System Extensions $1,000's***	***2.4 mgd System Extensions $1,000's***
Structural				
2.0 mgd Syst-volume, gal	14,000	$1.5/gallon	$21	
2.4 mgd Syst-volume, gal	17,000	$1.5/gallon		$26
Civil		20% Struct	$4	$5
Pile Foundation	LS		$40	$40
Pumps & Motors				
2.0 mgd System	3 Sets	$17,000/set	$51	
2.4 mgd System	3 Sets	$18,000/set		$54
VFD's				
2.0 mgd System-60 hp	3	$17,000 ea	$51	
2.4 mgd System- 75 hp	3	$19,000 ea		$57
Valves	LS		$12	$12
Process Piping			$20	$20
Flow Meter	LS		$10	$10
Surge Tank	LS		$30	$30
Surge Tank Air Supply	LS		$10	$10
Electrical/Instrumentation (50% of Mech Equip.)	LS		$90	$95
Subtotal(1)			$340	$360
Add 25% Contingency			$85	$90
Subtotal			$425	$450
Add 15% Contractor OH & P			$65	$70
	Total Estimated Construction Costs		**$490**	**$520**

(1) Rounded to closest $10,000's

13.0 Summary of Estimated Costs of Benicia-To-Valero Recycled Water Conveyance System

The Recycled Water Conveyance System consists of two capacity scenarios, as follows:

- 2.0 mgd capacity, if the RO System is located at the City's WWTP
- 2.4 mgd capacity, if the RO System is located at Valero Refinery

Table 13.1 presents a summary of the estimated construction costs for the entire Recycled Water Conveyance System.

Table 13.1
Summary of Estimated Construction Costs of Recycled Water Conveyance System

System Component	*2.0 mgd Scenario $1,000's*	*2.4 mgd Scenario $1,000's*
Recycled Water Pipeline	$2,060	$2,060
Recycled Water Supply Pump Station	$490	$520
Total Estimated Construction Costs	**$2,550**	**$2,580**

14.0 Recommendation

Based on the analyses presented herein, CDM recommends that the City and PURE accept the recommendation to use a new, 14-inch pipeline for the conveyance system.

TM 4

City of Benicia-Water Reuse Project Draft Technical Memorandum No. 4 - Analysis of Facilities Siting Alternatives

To: *Chris Tomasik*

CC: *PURE Members*

DATE: *February 2, 2005*

Executive Summary

Development and Evaluation of Siting Alternatives

Three siting alternatives were developed, based on the location of major process treatment components. These alternatives are:

- Alternative No. 1 – All treatment facilities at Benicia WWTP (MF/RO/UV)
- Alternative No. 2 – MF and UV at the Benicia WWTP and the RO system at Valero
- Alternative No. 3 – MF at the Benicia WWTP and the RO and UV systems at Valero

Flow design criteria were established for the three alternatives, depending on where the facilities are located. MF and RO have the same flow criteria regardless of location (those being 2.5 mgd and 2.3 mgd, respectively.) The Recycled Water Pump Station and the UV system have different design capacities, depending on if the UV is located before or after RO. Also, an intermediate Recycled Water Supply Pump Station is required at Valero, owing to the location of the facilities to be located at Valero, as proposed by Valero staff.

Electrical power supply requirements were developed for each alternative and integrated into the analysis. For operating and maintenance costs, electrical power demands were estimated for each alternative and were used to calculated present worth values.

The results of present worth analysis is as shown in Table E-1.

Table E-1
Summary of Present Worth Analysis of Siting Alternatives

System Component	Alt No. 1 MF/RO/UV @ Benicia	Alt No. 2 MF/UV @ Benicia and RO @ Valero	Alt. No. 3 MF @ Benicia and RO/UV @ Valero
	$1,000's	$1,000's	$1,000's
Estimated Construction Costs (from Table 3.1)	$780	$1,380	$1,380
Add 35% for Engineering and CM	$270	$480	$480
Estimated Capital Costs	$1,050	$1,860	$1,860
Total Estimated Annual O&M Costs (from Table 3.2)	Base	$56	$56
Present Worth of O&M Costs(1) (2)	Base	$640	$640
Total Estimated Difference in Present Worth Values(3)	**$1,050**	**$2,500**	**$2,500**

(1) Rounded to closes $10,00s
(2) 20 Years @ 6% interest

Based on the analysis locating the entire Water Reuse Treatment System at the Benicia WWTP appears to be the most cost-effective Siting Alternative. Three major factors that could change this analysis are as follows:

- Location of the treatment components at Valero near the cooling towers at Elevation 95.
- Further analysis of the electrical supply conditions at Valero including the need for standby power there.
- Results of the toxicity testing of the RO concentrate with Valero's and Benicia's effluent. Availability of dilution ratios and toxicity results could dictate the location.

Development and Evaluation of Flow Equalization Alternatives

From the results of prior TM's, it has been determined that the proposed Water Reuse Treatment System will require a steady flow of 2.5 mgd, allowing for a reject stream, in order to deliver the design flow of 2.0 mgd recycled water to Valero. Based on an analysis of City flow data it was determined that approximately 400,000 gallons of storage is required to equalize plant flow and provide the 2.5 mgd continuous flow. Owing to the existence of the City's Multi Purpose Basins (MPBs), which have a total storage capacity of one million gallons, it was deemed appropriate to determine if a portion of them could be utilized for flow equalization. Plant operations staff advised that this may be a possibility, provided other functions could continue to be served, including emergency storage and wet weather storage.

The City's existing Reservoir R1 has been suggested for use as storage for recycled water. However, since it has been determined that diurnal storage of either primary or secondary effluent is most beneficial, this tank is too far away to be of such use. Also, it is too distant from the recycled water transmission line to be of use in storing product water.

Concerning wet weather storage, the City is completing improvements to the treatment plant to accommodate wastewater flows through the plant during storm events up to 20-year return frequency. This involves a combination of treatment and temporary storage for flows greater than 12 mgd. Therefore, it is important to have the MPBs available for large storm events. When those events occur, there will be adequate minimum flow of secondary effluent and storage for equalization would not be required.

Hence, three alternatives for flow equalization were developed as follows:

- Alternative No. 1 – Equalize flow by storing primary effluent in the MPBs
- Alternative No. 2 – Equalize flow by storing secondary effluent in the MPBs
- Alternative No. 3 – Equalize flow by storing secondary effluent in a new, 400,000 gallon storage tank

Construction and capital costs were estimated along with power operating costs. A present worth (life cycle) analysis was performed to determine which alternative is most economical over a 20-year period. Alternative No. 2 was estimated to have a PW value nearly 30% less than the Alternative No. 1. Alternative No. 3 was estimated to be the most costly. Hence, CDM recommends that Alternative No. 2 be selected as the preferred Flow Equalization method.

The total construction cost of Alternative No. 2 was estimated at approximately $300,000. There may an opportunity to reduce this cost based on recent discussions with plant staff by eliminating a new pump station.

Development and Evaluation of Electric Power Supply Alternatives

Two electric supply alternatives were developed and evaluated based on primary power supply from PG&E. Those alternatives are:

- Alternative No. 1 – Power Supply through Existing Power Supply at the Plant
- Alternative No. 2 – New PG&E Power Service for the Water Reuse Treatment System

An estimate of the electrical loads for the new Water Reuse Treatment Plant as well as improvements to the existing Benicia WWTP, was made. Based on the estimated new loads, it was determined that if the entire WRTP is located at the Benicia WWTP, the existing electrical transformer is adequate, but a new feeder circuit breaker in the existing switchgear would be required. Cabling from the existing Blower Building easterly to the site of the new WRTP would also be required. An alternative of providing a new 750 kW service connection, specifically for the Project, was posed to PG&E. The proposed new service would come off an existing power line in East "G" street and into the WWTP at the NE corner of the site. As of this writing, PG&E has not provided any conceptual cost estimates for such a new service. Therefore, for purposes of the siting analysis, it was assumed that new circuit breaker and cabling would be required.

The history of power outages at the WWTP over the last three years was reviewed. Although supply has been highly reliable (one lasted about seven hours and five lasted between 50 and 80 minutes), future outages in the form of brownouts can be expected. Hence, the criterion of providing 100% standby power supply has been established.

The existing standby power generating system at Benicia was evaluated and found to be inadequate if the entire WRTP is located at the Benicia WWTP. Hence, a new, 500kW generator would be required under siting Alternative No. 1. For the other siting alternatives, the existing standby generator was determined adequate for the estimated new electrical loads at Benicia for the Water Reuse Project as well as existing and future loads for the basic secondary treatment plant.

Valero was contacted to gain a preliminary understanding of the existing electrical power supply in the vicinity of the proposed location of the Water Reuse Treatment facilities that would be located at Valero under siting Alternative No. 2 & 3. New 4,160 kVA cabling, transformer and switchgear would be required at Valero.

Overview of Alternative Energy Sources

Alternative energy sources are resources that are constantly replaced and are usually less polluting than those derived from the burning of fossil fuels. Alternative energy sources include: biomass, geothermal, hydroelectric, solar, wind and ocean.

Biomass is renewable energy that is produced from organic matter. Biomass fuels include: wood and forest and mill residues, animal waste, grains, agricultural crops, aquatic plants and organic sludge from wastewater treatment plants.

The City's WWTP stabilizes the biomass removed from the process by anaerobic digestion. This process converts organic material to methane gas, which is used in a boiler to heat the biomass (sludge) to sustain the process.

Geothermal energy uses heat from within the earth. Wells are drilled into geothermal reservoirs to bring the hot water or steam to the surface. The steam then drives a turbine-generator to generate electricity in geothermal plants.

Hydroelectric energy employs the force of falling water to drive turbine-generators to produce electricity. Hydropower produces more electricity than any other alternative energy sources.

Solar energy is generated without a turbine or electromagnet. Special panels of photovoltaic (PV) cells capture light from the sun and convert it directly into electricity. The electricity is stored in a battery.

The conceptual cost of an On-Grid, PV system would be in the $7 to $8 per installed Watt range. This unit cost translates to approximately $5 million for a capacity of 750kW. Costs for an at-ground structural system to support the PV panels would be in addition to the $5 million. Also, a system with an out-put capacity of 750 kW would require approximately 83,000 sf of panels, or about two acres. This is an area approximately 300 ft by 300 ft. Even though there maybe significant rebates (up to 40%) from the California Energy Commission for solar systems in this size range, still the space limitations at the City's WWTP preclude this alternative energy source from further consideration.

Wind energy can be used to produce electricity. As wind passes through the blades of a windmill, the blades spin. The shaft that is attached to the blades turns and powers a pump or turns a generator to produce electricity. Electricity is then stored in batteries. The speed of the wind and the size of the blades determine how much energy can be produced. In the size range of 600kW to 1,000kW, a conceptual installed unit cost for wind turbines is approximately $800 to $1,000 per kW. Hence, for a wind turbine system that would supply 750 kW of electricity, the installed cost would be approximately $0.9 million. In the size range stated, the height of these wind turbines is approximately 150 feet. Owing to the high installed costs and potential impacts, including visual, raptors and noise, wind power is not considered a feasible supplemental power supply for the City's Water Reuse Project.

Ocean Energy contains both thermal energy from the sun's heat and mechanical energy from tides and waves. Ocean thermal energy conversion (OTEC) converts solar radiation to electric power. OTEC power plants use the difference in temperature between warm surface waters heated by the sun and colder waters found at ocean depths to generate electricity. These systems are in the experimental stage and are being considered on a large scale.

In September 2000, Assembly Bill 970 was approved, which called for the creation of more energy supply and demand programs. As a result, in March 2001, the California Public Utilities Commission (CPUC) issued a decision creating the Self-Generation Incentive Program (SGIP) to offer financial incentives to their customers who install certain types of distributed, self-generation facilities to meet all or a portion of their energy needs, up to 1.5 MW, although the maximum incentives basis remains capped at 1,000 kW. Incentives range from $1.00/Watt to $4.50/Watt depending on the type of technology used and the type of fuel or renewable energy source.

The energy needs of the Water Reuse Project could also be supplemented by renewable systems that can burn digester (methane) gas. These systems include fuel cells, micro-turbines and generators driven by internal combustion (IC) engines.

Fuel cells are electrochemical devices that combine hydrogen fuel and oxygen from the air to produce electricity, heat and water. Fuel cells operate without combustion, so they are virtually pollution free. However, to operate on digester gas, the gas must be scrubbed of hydrogen sulfide prior to injection into the unit.

Micro-Turbines are similar to small jet engines that burn either natural gas or biogas (digester gas). Specially designed micro-turbines that burn biogas are provided with emission controls that result in emissions with significantly less NOx and other air pollutants than those from reciprocating engine generator sets.

Internal combustion engines, driving electrical generators, can run on natural gas, digester gas or a blend of the two. In order to be permitted by the AQMD, engines must be of the "clean burn" type, which generally come in the size of 1 mW and larger. Since there is considerable heat lost from an IC engine, the heat is usually recovered for purposes as heating digester sludge and/or buildings. This type of system is called Cogeneration, of CoGen.

The estimated installed costs of these three types of renewable fueled systems range from $700 to $2,000 per installed kW. The amount of digester gas available at the City's WWTP would dictate the size of system that could be implemented. A thorough process analysis of the digester system and an economic analysis would be required to determine the feasibility of implementing such a system.

1.0 Introduction and Purpose of the Technical Memorandum

A joint Water Reuse Project is being undertaken by the City of Benicia and the Valero Refinery to supply approximately 2 mgd of recycled water for cooling water make up at the Refinery.

TM 1, dated September 2004, evaluated alternative treatment processes to meet Valero's cooling water mineral requirements. The results of the evaluation were that biological ammonia removal followed by the MF/RO process is the applicable water reuse treatment system to meet Valero's water quality requirements.

TM 2, dated 4 November 2004, evaluated alternative technologies for disinfection of recycled water produced by the micro-filtration and reverse osmosis process. The result of the evaluation was that the low-pressure, high-intensity UV process is the best disinfection system to meet regulatory requirements and Valero's water quality requirements. TM 2 also contained discussions that disinfection should be located at Benicia as it may present less risk, in the event of a pipeline leak, to convey recycled water that has been disinfected and meets Title 22 requirements for unrestricted contact. However, within the City's wastewater collection system, there are pump stations that convey raw sewage. Albeit these are buried pipes. Hence, consideration should be given to siting the UV and RO at Valero.

TM 3, dated 9 November 2004, contained a conceptual design of a complete conveyance system for recycled water from the Benicia WWTP to the Valero Refinery Cooling Towers. The economics of rehabilitating Valero's existing pipeline compared with constructing a new pipeline were evaluated for the off-site portion of the conveyance pipeline. The results of the evaluation were that installing a new, 14-inch diameter pipeline is more cost-effective than rehabilitation. Also, a new pipeline has fewer unknowns and less risk. Hence, a new 14-inch diameter pipeline was selected for the conveyance system.

1.1 TM4

The purpose of this TM is to evaluate the economics of locating the water reuse treatment facilities on either the Benicia WWTP site or on the Valero Refinery site. Owing to the ongoing status of toxicity testing, it cannot yet be determined if the impacts of concentrate disposal will influence the final decision on location of the RO system.

Other project components evaluated and discussed include requirements for power supply, alternative energy supply and diurnal storage facilities to accommodate variations in wastewater supply as input to the Water Reuse Treatment System.

TM4 is composed of the following major sections:

- Overview of Siting Alternatives
- Conceptual Design of Siting Alternatives
- Economic Evaluation of Siting Alternatives
- Preliminary Recommendation for Location of the Water Reuse Treatment System
- Development and Evaluation of Flow Equalization Alternatives
- Development and Evaluation of Electrical Utility Power Supply Alternatives
- Review of Alternative Energy Sources

2. Siting of the Water Reuse Treatment Facilities

2.1 Overview of Siting Alternatives

The Water Reuse Treatment Facilities can be located at either the Benicia WWTP or at the Valero Refinery. Adequate space is available at either location. The basic flow criterion for the overall Water Reuse Project is to deliver 2.0 mgd of recycled water to the Valero cooling towers. However, there are different flow criteria for various components of the overall water reuse system, depending on where each process component is located. Three siting alternatives have been developed, and Table 2-1 lists the alternatives and the flow criteria for the various project components under each alternative.

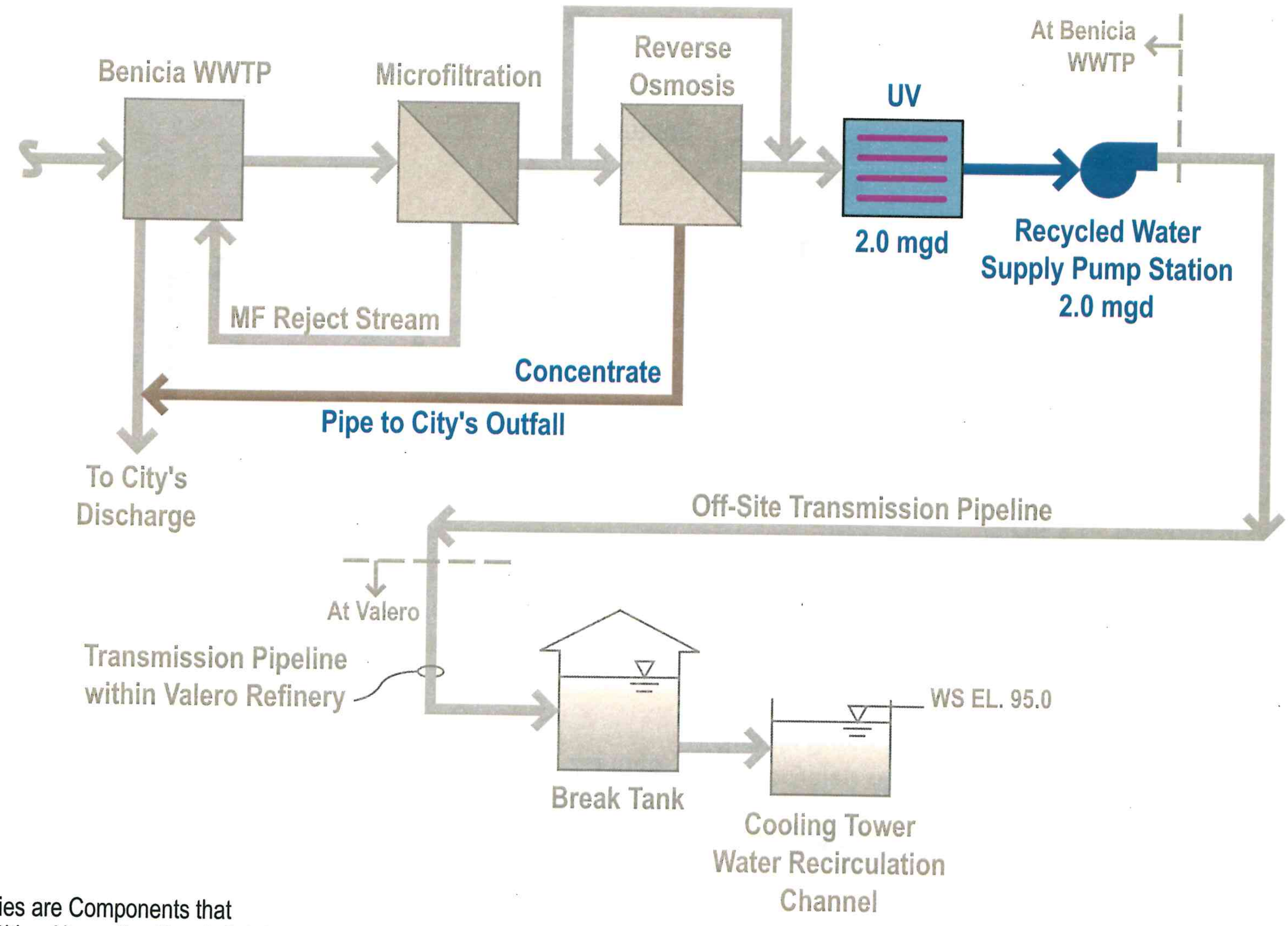
Table 2-1 **Flow Criteria for Project Components**			
System Component	***Alternative No. 1 MF/RO/UV @ Benicia***	***Alternative No. 2 MF/UV @ Benicia and RO @ Valero***	***Alternative No. 3 MF @ Benicia and RO/UV @ Valero***
MF (Feed Water)	2.5	2.5	2.5
UV	2.0	2.3	2.0
RO (Feed Water)	2.3	2.3	2.3
Recycled Water Supply Pump Station (RWSPS)	2.0	2.3	2.3
Intermediate Recycled Water Supply Pump Station (IRWSPS)	Not required	2.0	2.3

Figure 2-1 presents a schematic diagram of the overall Water Reuse System with the MF/RO/UV System located at the Benicia WWTP. Figure 2-2 presents a schematic diagram of the overall Water Reuse System with the MF/UV components at Benicia and the RO system located at the Valero Refinery. Figure 2-3 presents a schematic diagram of the overall water reuse system with the MF at Benicia and the RO/UV at Valero. Also, as shown in Table 2-1, both the MF and the RO systems have the same design capacities under all three alternatives. In addition to the notable differences in flow capacities for UV and the RWSPS, Alternatives 2 and 3 require for an Intermediate Recycled Water Supply Pump Station (IRWSPS) and additional pipelines. Figure 2-4 shows a conceptual plan of the WRTP at the Benicia WWTP. Figure 2-5 shows a conceptual plan of the RO System, the UV system, and IRWSPS at Valero. Figure 2-6 contains a conceptual site plan with the RO and the UV at Benicia and the IRWSPS at Valero. Treatment processes at Valero would be tentatively located near the Valero WWTP, in accordance with guidance from Valero's Steve Penny.

2.2 Summary of Related Components from TM 2 and TM 3

As shown in Table 2-1, the UV system is about 15% larger for Alternative No.2 (RO at Valero) since it would precede the RO component, rather than following it. The increased capacity is related to the 15% reject rate of the RO system. (Please refer to TM 1 for more details.) Hence, the costs, both capital and O&M, for the larger system must be included in the siting analysis. These cost items, developed in TM 2 for the UV system, will be included in the analysis below.

The RWSPS would also need to be 15% larger in capacity to account for the 15% reject in order to the RO system to produce a permeate flow of 2 mgd. Hence, the costs, both capital and O&M, for the larger RWSPS must be included in the siting analysis. These cost items, developed in TM 3 for the conveyance system, will be included in the analysis below.



Note:
Colored Facilities are Components that Differ Among Siting Alternative Nos 1, 2 & 3.

Figure 2-1
Benicia Water Reuse Project
Facilities Siting Alternative No.1 Process Schematic for MF/RO/UV at Benicia WWTP

CDM

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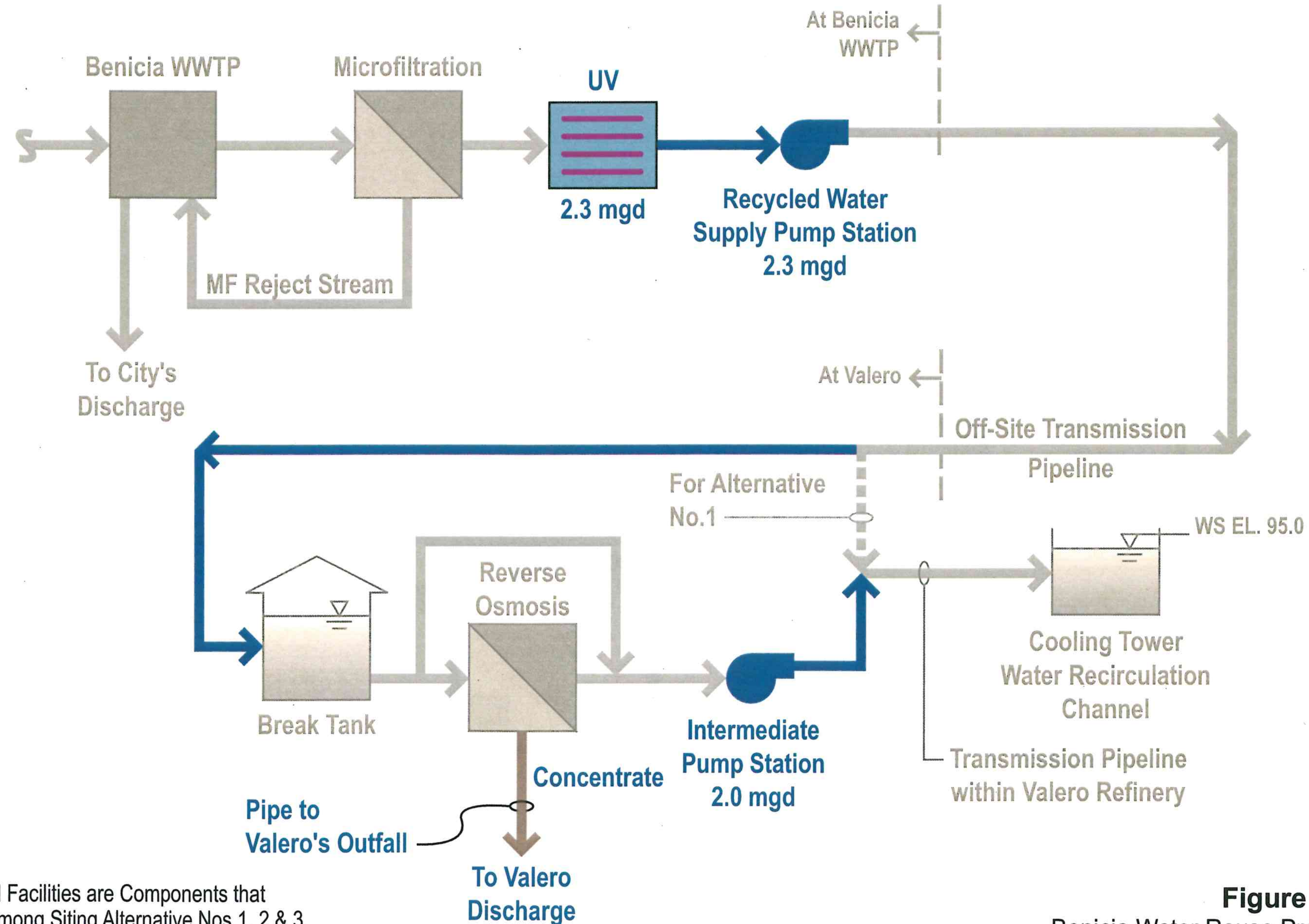
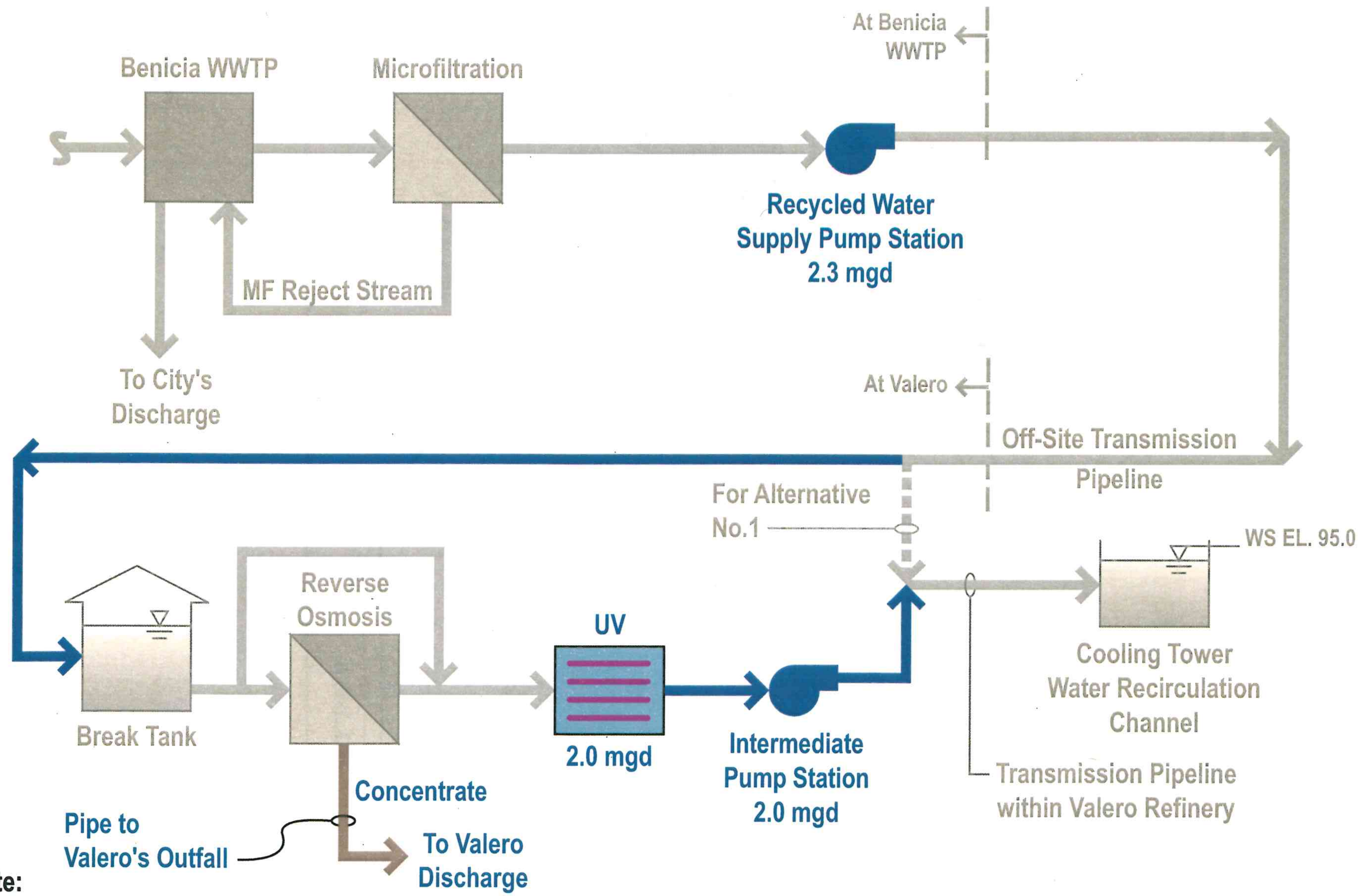


Note:
Colored Facilities are Components that
Differ Among Siting Alternative Nos 1, 2 & 3.

Figure 2-2
Benicia Water Reuse Project
Facilities Siting Alternative No.2 Process Schematic for
MF/UV at Benicia WWTP and RO at Valero Refinery

CDM



Note:
Colored Facilities are Components that Differ Among Siting Alternative Nos 1, 2 & 3.

Figure 2-3
Benicia Water Reuse Project
Facilities Siting Alternative No.3 Process Schematic for MF at Benicia WWTP and RO/UV at Valero Refinery

CDM

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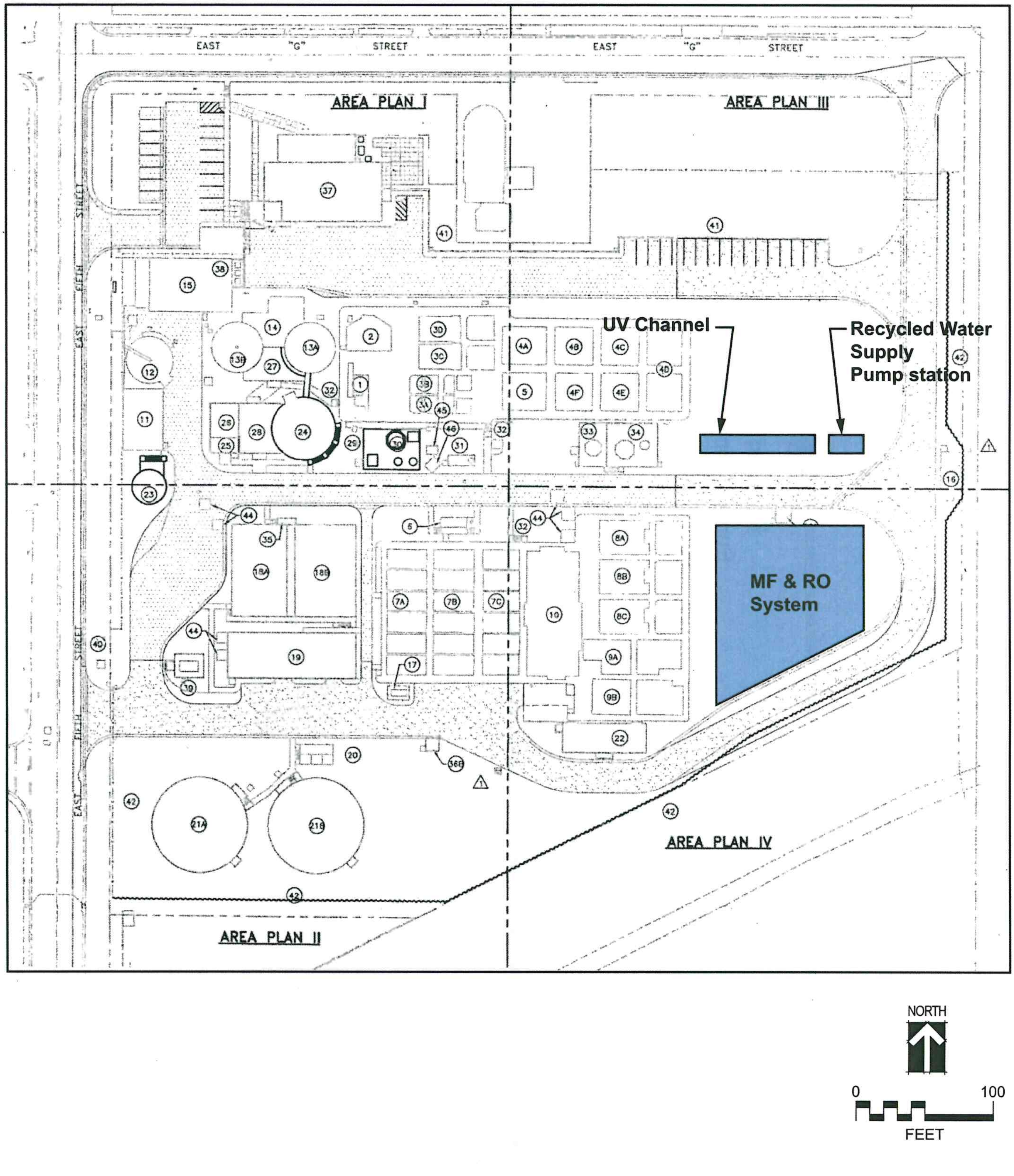
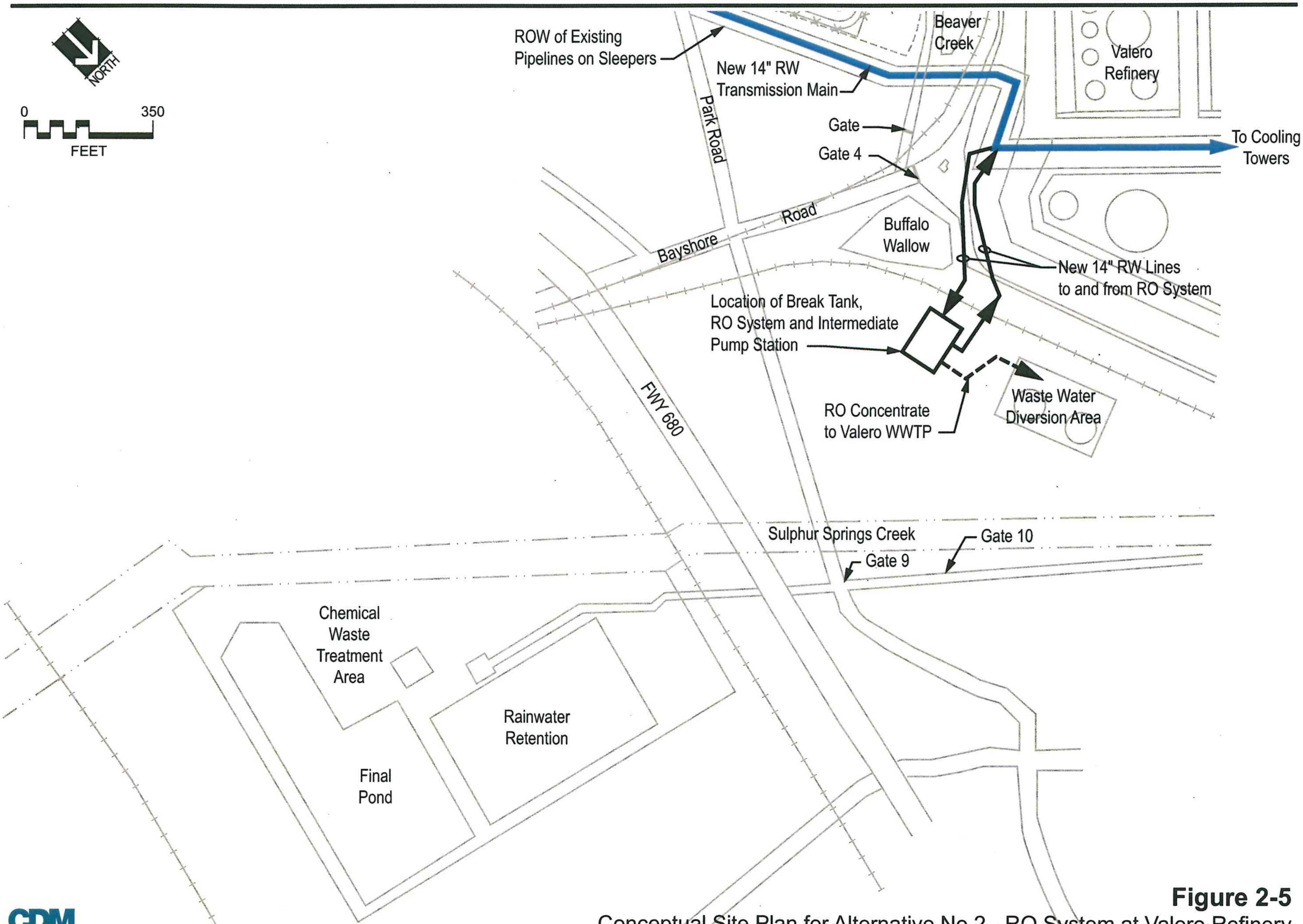


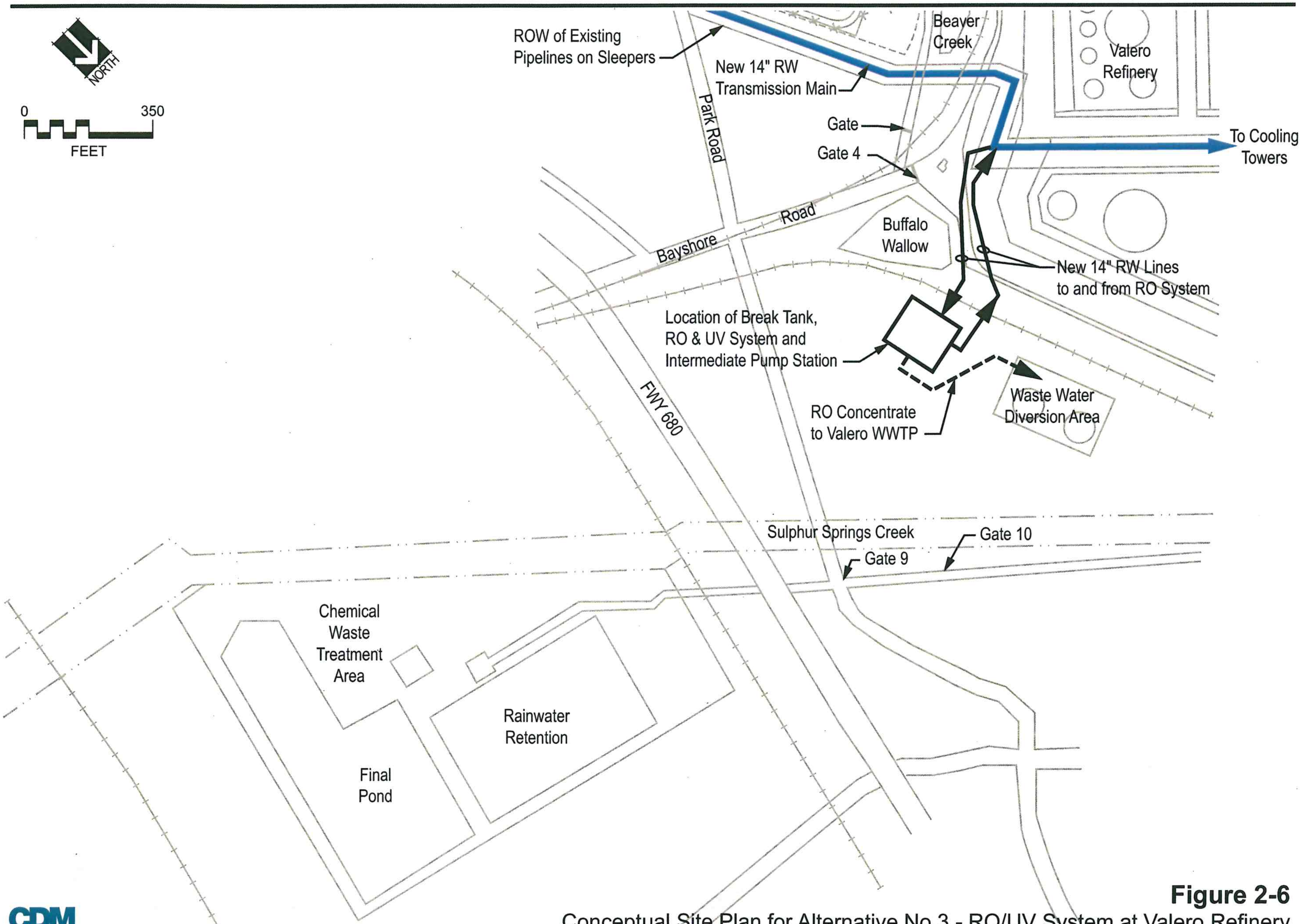
Figure 2-4
Conceptual Site Plan for RO System at Benicia

CDM



CDM

Figure 2-5
Conceptual Site Plan for Alternative No.2 - RO System at Valero Refinery



CDM

Figure 2-6
Conceptual Site Plan for Alternative No.3 - RO/UV System at Valero Refinery

2.3 Additional Project Components for the Siting Alternatives

Components added to the applicable siting alternatives include the additional piping to and from the RO system at the Valero Refinery, concentrate disposal lines, which are different for each alternative and the addition of the Intermediate Recycled Water Supply Pump Station, which would pump the RO permeate (recycled water) to the cooling towers.

2.4 Conceptual Design of Intermediate Recycled Water Supply Pump Station (IRWSPS)

2.4.1 Physical Description

The Intermediate Recycled Water Supply Pump Station (IRWSPS) will receive flow from the Reverse Osmosis (RO) System, located at the Valero Refinery, as shown in Figure 2-3. A cast-in-place concrete structure with a lower level clearwell is recommended. The pumps will be vertical turbine (or mixed-flow) type, mounted outdoors on a concrete deck over the clear well. Electrical equipment for the pumps will be housed in the electrical room of the control building for the RO system. Recycled water will flow from the RO system into the IRWSPS clear well from where it will be pumped into the recycle water conveyance system, mounted on sleepers within the refinery. The pump motors, discharge piping and valves, and monitoring and sampling equipment will be located in the deck area over the clear well. The pump station will be rectangular in shape with the plan dimensions being determined based on pump and other equipment space requirements in the pump deck area and to a secondary extent, storage volume in the clear well.

2.4.2 Pump Selection

Preliminary pump selection is based on the elevation difference between the IRWSPS and the cooling towers and the pipeline friction loses between the two points. The main pipeline, which parallels Avenues "E" South and "F", would be a 14-inch diameter pipeline, as described in TM-3. Because there will be two RO banks with 50% capacity, it makes sense to provide pumps that can easily meet one half the system design capacity. CDM's preliminary recommendation is that all pumps will be variable speed to provide for variations in demand. Pump equipment vendors were contacted to determine preliminary pump selection. The number of pumps and variable speed versus constant speed will be evaluated further in the preliminary design phase.

The hydraulic profile of the transmission line from the IRWSPS to the cooling tower water recirculation channel, will establish the static lift and frictional losses. The maximum water level in the IRWSPS clear well is assumed at the surrounding grade, Elevation 0.0. The bottom elevation of the clear well will be established based on the required operating level differential relative to the high level, minimum pump submergence and vertical distance between the clear well and the pump intake. The clear well will be provided with concrete baffle walls in accordance with Hydraulic Institute Standards to improve pump performance and avoid vortexing. The discharge elevation at the cooling towers is assumed to be Elevation 95, as shown in Figure 4-2 of TM-3.

2.4.3 Mechanical Design Considerations

2.4.3.1 Surge Control

A hydraulic transients analysis (surge) of the conveyance system, including identification of alternative mitigation measures, will be performed during final design. The analysis will define high and low pressures that could occur in the pipeline system and, if not mitigated, could damage the pipeline. Preliminary indications are that a surge tank will not be required at the IRWSPS; however, for budgetary purposes, a surge tank has been included.

2.4.3.2 Valves and Appurtenances

Each pump discharge will have a manual isolation butterfly valve and a check valve. It is anticipated that the check valve will be the double-door, fast-acting, silent type.

A manual isolation butterfly valve will also be provided on the discharge header downstream of the flow meter to isolate the meter from the transmission line.

Each pump discharge will also have an air release valve to release air on pump start-up. The air release valve will be specially designed for use on vertical turbine pumps and will contain an air pressure release throttling mechanism. Air release valves will also be provided on the pump discharge header at high points where air may accumulate.

2.4.3.3 Electrical Design Considerations

Power supply to the motors will be 480-Volt, 3-phase, 60-Hz power fed from a motor control center (MCC) located in the new RO equipment building.

2.4.4 Instrumentation, Monitoring and Control Design Considerations

2.4.4.1 Pump Control

The IRWSPS pumps will be automatically controlled by the level in the wet well. The IRWSPS will pump what comes into the wet well.

Control interlocks with other systems will be as follows:

- All of the IRWSPS pumps will be automatically stopped on low level in the wet well.
- All of the IRWSPS pumps will be automatically stopped on high level in the cooling tower make-up water channel to avoid overfilling the channel.
- All of the IRWSPS pumps will be automatically stopped on detection of critical alarm conditions at any of the upstream treatment processes.
- Under any of the hydraulic or process performance alarm conditions that would shut down the pumps, the recycled water would be routed to the Valero outfall until the alarm conditions have been addressed and cleared. There will need to be communication links between the RO system at Valero and the MF/UV system and RWSPS at Benicia. These details will be determined during the design phase.

2.4.4.2 *Monitoring*

The following are motoring provisions:

- Water level in the clear well will be continuously monitored using an ultrasonic level sensor, with separate float switches for high and low level alarms in the event of failure of the level sensor. The water level signal will be used for pump control as described above.

- A magnetic flow meter will be provided on the pump discharge header to measure pump flow rate. The flow signal will be used for regulatory and recycled water inventory record keeping, for IRWSPS monitoring, and for pump control as described above.

- A pressure transducer will be provided on the recycled water discharge header to continuously measure header pressure for the purposes of monitoring pump operation and head conditions in the transmission system.

- A locally indicating pressure gauge will be provided on the discharge header and on each pump discharge.

2.4.4.3 *Equipment Protection*

The following equipment protection measures will be considered in the design phase:

- Monitoring of motor winding and bearing temperature with automatic pump shut down on high temperature condition.

- Providing pumps with vibration monitoring and automatic pump shut down on high vibration condition.

- A discharge flow switch for each pump or check valve position monitor to confirm that flow is actually occurring. No flow would generally indicate that the pump is spinning at near shut-off head.

2.4.5 Design Criteria

Preliminary design criteria for the IRWSPS are presented in Table 2-2.

Table 2-2 ***Design Criteria - Intermediate Recycled Water Supply Pump Station***		
Criteria	***Units***	***2.0 mgd System***
System Pumping Requirements		
Design Capacity	mgd	2.0
Design Capacity	gpm	1,400
Design TDH	ft	130
Static Head	ft	100
Pumping Units		
Type	Vertical Turbine or Mixed Flow	
Number, Total/Duty/Standby		3/2/1
Design Capacity per Pump		700
Design TDH per Pump	ft	130
Min. Efficiency at Design Point	%	82
Stages per Pump	No.	3
Pump Operation		Variable
Minimum Speed	rpm	TBD
Pump Motors		
Type		TEFC
Size, each unit	hp	30
Drive Type		VFD
Synchronous Speed	rpm	1,800
Power Supply		480-V/3-phase/60Hz
Pump Discharge Piping		
Diameter	inch	8
Velocity at Design Flow	fps	4.43
Pumps Discharge Header Piping		
Diameter	inch	14
Velocity at Design Flow	fps	2.90
Discharge Flow Metering		
Type		Magnetic
Size	inch	10
Velocity at Design Flow Rate	fps	5.67

2.4.6 Estimated Construction Cost

The estimated construction cost for the IRWSPS were based on the following assumptions, unit costs and budgetary quotes from vendors:

- **Foundations** – Owing to the poor soil conditions (Bay mud) in the area available for the project, it will be necessary to place new structures on pile foundation systems. (Per communication with Steve Penny of Valero.) Foundation design assumptions used were the same as those for the RWSPS, discussed in TM-3. Hence, pre-cast concrete piles, driven to an approximate depth of 70 feet have been assumed. Conceptual design estimates were made for the number of piles per structure, and for mobilization and demobilization. Pile driving costs were assumed at $40/foot of pile, including the cost of the pile. Estimates were based on budget quotations obtained from a local pile driving subcontractor. During preliminary design, floating slab foundations will be considered.

- **Structural** – Pump station wet wells were assumed to be constructed of cast-in-place reinforced concrete and were estimated at $1.50/gallon capacity. Structural costs include excavation, reinforced concrete and structural backfill.

- **Civil** – Civil site work costs were estimated at 20% of structural costs (excluding foundation costs) to cover site preparation, grading, paving and site piping.

- **Mechanical –** Mechanical equipment costs were obtained from vendors and/or were based on experience from other similar projects. Budgetary costs for pumps and motors were obtained from J.M. Squared Associates and for VFD's, from Robicon.

- **Electrical** – Power supply will be required for the pump motors and related components. Electrical costs were estimated at 30 percent of the mechanical equipment cost based on experience with construction of similar systems.

- **Instrumentation** - Instrumentation will be required for process monitoring and control and for connection to the plant SCADA system. Typical instrumentation includes monitoring of flow rate, wet well water level, pump operating conditions, and others. The instrumentation costs are estimated at 20 percent of mechanical equipment cost.

Contractor's overhead and profit are included at 15 percent. Owing to the level of detail developed in this conceptual design phase a contingency allowance of 25 percent is included to account for lack of detailed information, estimating inaccuracies, and relatively small items that may not have been included.

Based on the design criteria and bases of cost estimates, presented herein, a conceptual cost estimate was developed for the IRWSPS for the design flow of 2.0 mgd, discussed above. The cost estimate is contained in Table 2-3.

Table 2-3
Estimated Construction Cost – 2.0 mgd Intermediate Recycled Water Supply Pump Station

Items	*Quantities*	*Unit Prices ($/unit)*	*2.0 mgd System Extensions $1,000's*
Structural	14,000 gal	$1.5/gallon	$21
Civil		20% Struct	$4
Pile Foundation	LS		$40
Pumps & Motors	3 Sets	$13,000/set	$39
VFD's	3	$10,000 ea	$30
Valves	LS		$12
Process Piping	LS		$20
Flow Meter	LS		$10
Surge Tank	LS		$30
Surge Tank Air Supply	LS		$10
Electrical/Instrumentation (50% of Mech Equip.)	LS		$75
Subtotal[(1)]			$290
Add 25% Contingency			$70
Subtotal			$360
Add 15% Contractor OH & P			$50
	Total Estimated Construction Costs		**$410**

(1) Rounded to closest $10,000's.

2.4.7 Estimated O&M Costs for the IRWSPS

Operations and maintenance costs must also be considered in the analysis since this is an addition system that will consume electricity and require operator attention. Electrical power unit cost was assumed at $0.12 per kWhr, although Valero may have its own electrical generation and distribution system. Approximately 400,000 kWhr per year will be required to pump the recycled water to the cooling towers from the RO system location, shown in Figure 2-3. This power draw would cost approximately $48,000 per year.

3. Economic Analysis of Siting Alternatives

3.1 Summary of Estimated Construction Costs of Siting Alternatives

Project components that differ in capacity and hence estimated capital and operating costs are listed in Table 3-1, along with components unique to the three alternatives, such as the Intermediate Pump Station at Valero. Estimated costs for electric power supply were brought forward from Section 5, which addresses electric power supply alternatives and costs. As stated in Section 5, PG&E has not yet provided estimated costs for the alternative of a new service for the project. Hence, costs for electric power supply are tentative. Total estimated construction costs in this table are not total estimated project costs for all aspects of site work. They represent only the different features between the three alternatives.

Similar to construction cost estimates, Table 3-2 presents operating costs for equipment and components that are different for each alternative. Common components have not been included because they tend to obscure the cost differences between alternatives.

Table 3-1
Summary of Estimated Construction Costs of Siting Alternatives

System Component	***Alt No. 1 MF/RO/UV @ Benicia***	***Alt No. 2 MF/UV @ Benicia and RO @ Valero***	***Alt. No. 3 MF @ Benicia and RO/UV @ Valero***
	$1,000's	***$1,000's***	***$1,000's***
Additional Recycled Water Pipelines			
460 ft of 14-in pipe on over head sleepers Assumed Unit Cost of $110/ft, including sleepers	Not applicable	$50	$50
Concentrate Disposal Lines			
At Benicia: 200ft of 6-in pipe @ assumed unit cost of $50/ft	$10	n.a.	n.a.
At Valero: 300ft of buried 6-in pipe @ assumed unit cost of $50/ft	n.a.	$15	$15
UV Disinfection Systems	Same cost for both, even though design flows are different (Refer to TM 2)		
Recycled Water Supply Pump Station	$490	$520	$520
Intermediate Pump Station @ Valero	Not applicable	$410	$410
Electrical Power Supply (see Table 5-2)	$280	$380	$380
Total Estimated Construction Costs	**$780**	**$1,375**	**$1,375**

n.a. = not applicable

Table 3-2
Summary of Estimated Annual O&M Costs of Siting Alternatives

System Component	Alt No. 1 MF/RO/UV @ Benicia	Alt No. 2 MF/UV @ Benicia and RO @ Valero	Alt. No. 3 MF @ Benicia and RO/UV @ Valero
	$1,000's	$1,000's	$1,000's
Recycled Water Supply Pump Station at Benicia – Additional Estimate Power Cost	n.a.	$4	$4
Intermediate Recycled Water Supply Pump Station @ Valero – Estimated Power Cost	n.a.	$48	$48
UV System – Estimated Additional Power Cost for 2.3 mgd System (Refer to TM 2)	n.a.	$4	$4
Total Estimated Annual O&M Costs	**Base**	**$56**	**$56**

Table 3-3 presents the summary of the present worth analysis. Alternatives No. 2 and 3 are estimated to have present worth values more than twice that of Alternative No. 1, which is all treatment components located at Benicia. The major component that has a very large impact on this analysis is the intermediate pump station. Not only is the capital cost a significant influence, but also the operating cost to lift the water 100 feet back up to the cooling towers at Elevation 95 results in a very large operating cost over 20 years.

If the water reuse treatment components, proposed to be located at Valero, were to be located near the cooling towers at or near Elevation 95, there would be significant difference in the present worth values between Alternative No. 2 and 3 and Alternative No. 1 because the IRWSPS would not be required. However, a RO concentrate disposal line would be required back to the existing Valero WWTP for disposal.

Based on the analysis with the water reuse treatment facilities located as shown in Figure 2-5, locating the entire water reuse treatment system at the Benicia WWTP appears more cost-effective than locating some of the process units at Valero.

Three major factors could change the results of this analysis, as follows:

- Location of the treatment components at Valero near the cooling towers at Elevation 95.
- Further evaluation of the electrical power supply requirements at Valero including the need for standby power there.
- Results of the toxicity testing of the RO concentrate with Valero's and Benicia's effluent. Availability of dilution ratios and toxicity results could dictate the location.

Table 3-3
Summary of Present Worth Analysis of Siting Alternatives

System Component	***Alt No. 1 MF/RO/UV @ Benicia***	***Alt No. 2 MF/UV @ Benicia and RO @ Valero***	***Alt. No. 3 MF @ Benicia and RO/UV @ Valero***
	$1,000's	***$1,000's***	***$1,000's***
Estimated Construction Costs (from Table 3.1)	$780	$1,380	$1,380
Add 35% for Engineering and CM	$270	$480	$480
Estimated Capital Costs	$1,050	$1,860	$1,860
Total Estimated Annual O&M Costs (from Table 3.2)	Base	$56	$56
Present Worth of O&M Costs(1) (2)	Base	$640	$640
Total Estimated Difference in Present Worth Values	**$1,050**	**$2,500**	**$2,500**

(1) Rounded to closes $10,00s
(2) 20 Years @ 6% interest

4. Evaluation of Flow Equalization Alternatives

4.1 Overview of Flow Equalization Alternatives

The rate of wastewater flow into the Benicia WWTP varies throughout the day. Seasonal variations also exist, based on wet weather conditions, primarily in the wintertime. Daily or "diurnal" flow varies from lows in the very early morning hours of about 1 mgd to peak flows in the range of 3.5 mgd up to 5 mgd. Figures 4-1 and 4-2 present typical diurnal flow curves for summertime and wintertime conditions, respectively.

As seen in Figure 4-1, there will be times when 2.5 mgd of secondary effluent is not available for advanced treatment and delivery to Valero. Two main options exist to overcome this flow variation situation. First, flow equalization of the existing treatment could be implemented; and second, the Water Reuse Treatment system could be designed to handle peak flows and the product recycled water could be equalized, either at the Benicia WWTP or at Valero. Under the latter option, the MF/RO/UV system would need to be substantially larger and more costly. Also, those processes do not lend themselves to operating under large flow variations. For these two important reasons, treating the peak flow option was not given consideration.

The required amount of flow equalization storage was estimated from the diurnal flow curves. Figure 4-3 shows a graphic estimate of the amount of storage required to achieve a continuous flow of 2.5 mgd of secondary effluent to "feed" the Water Reuse Treatment System. As shown in the figure, approximately 350,000 gallons of storage are required. A storage amount of 400,000 gallons is used to include a 10% contingency.

It should be noted that the average dry weather flow to the WWTP for the months of July, August, and September was 2.68 mgd. So, nearly all plant flow during this period would be devoted to supplying the project with secondary effluent.

4.2 Description of Wet Weather Facilities Operations

The existing WWTP has storage basins called Multi-Purpose Basins (MPBs) that have a total storage capacity of one million gallons. Hence, it is appropriate to consider the option of providing equalization within the existing treatment plant.

Before developing the alternatives for flow equalization, it is appropriate to present an overview discussion of the wet weather facilities and operations pertaining to the City's Wastewater Treatment Plant (WWTP). The WWTP is designed to handle a 20-year storm event, which translates to a peak flow of about 30 mgd influent to the Plant. In 2004, the City constructed a wet weather improvement project that, among other things, added a major relief sewer, control structures and a new wet weather screenings structure at the WWTP.

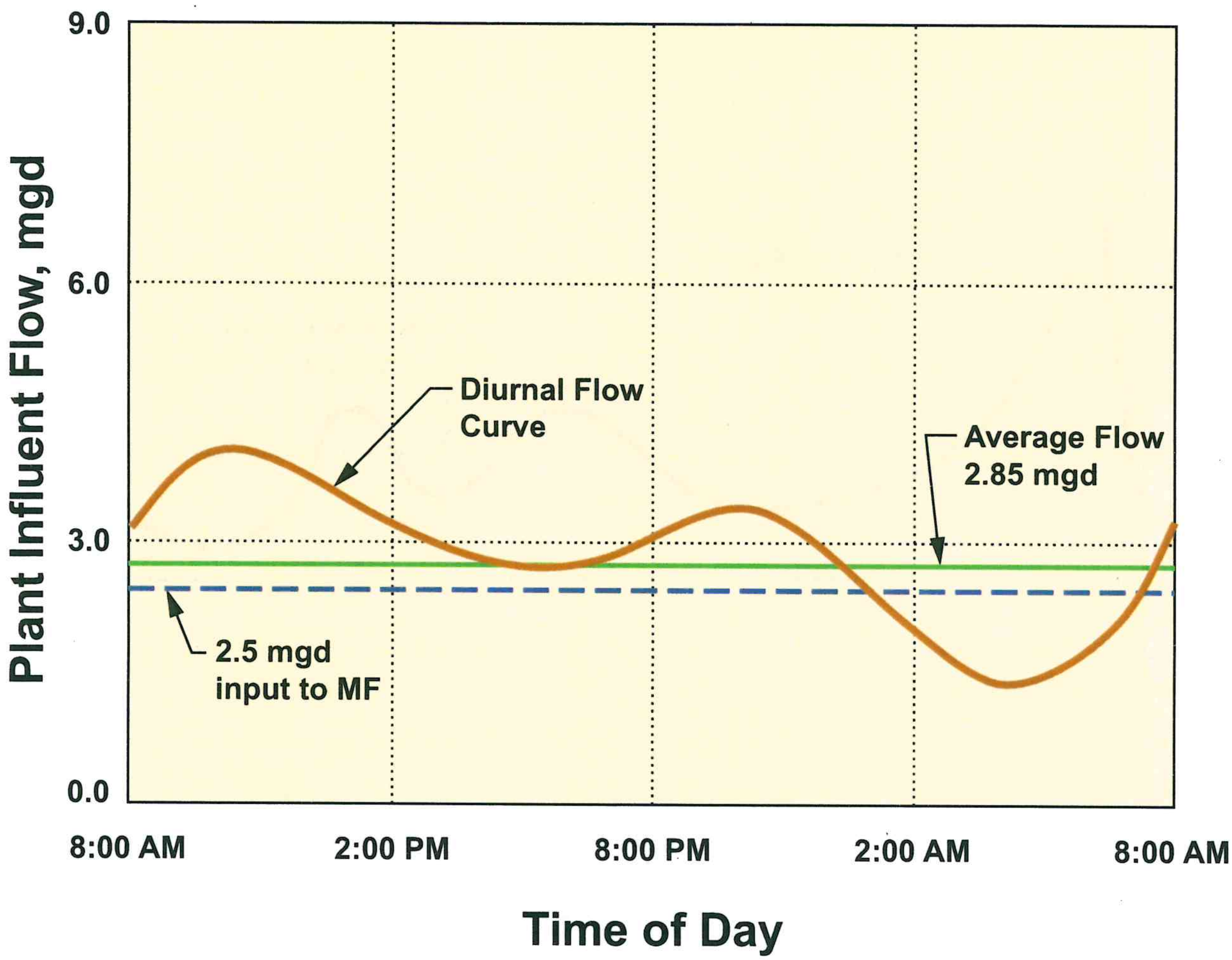


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Figure 4-1
Benicia WWTP - Typical Summertime Diurnal Flow Curve

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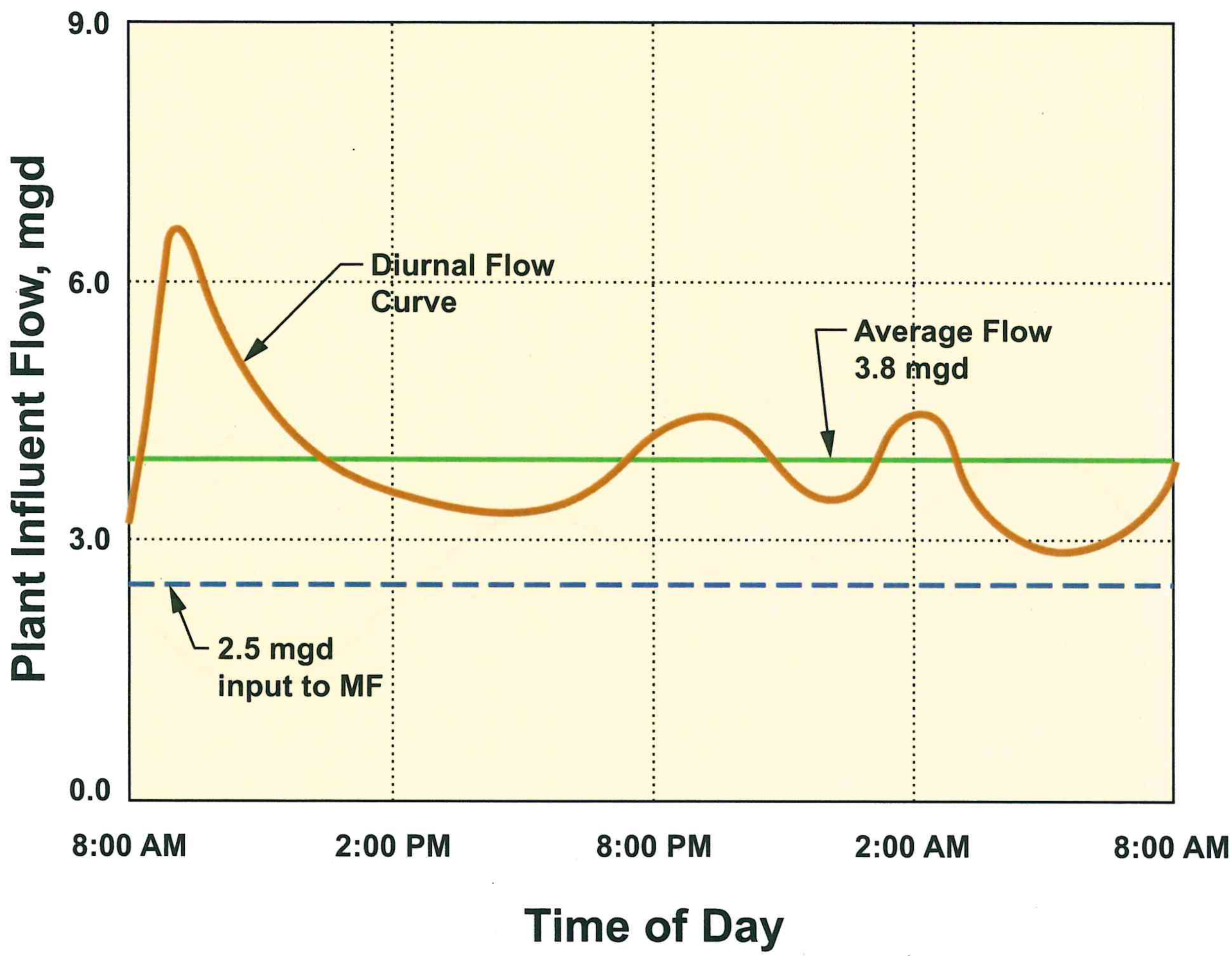

Figure 4-2
Benicia WWTP - Typical Wet Weather Diurnal Flow Curve

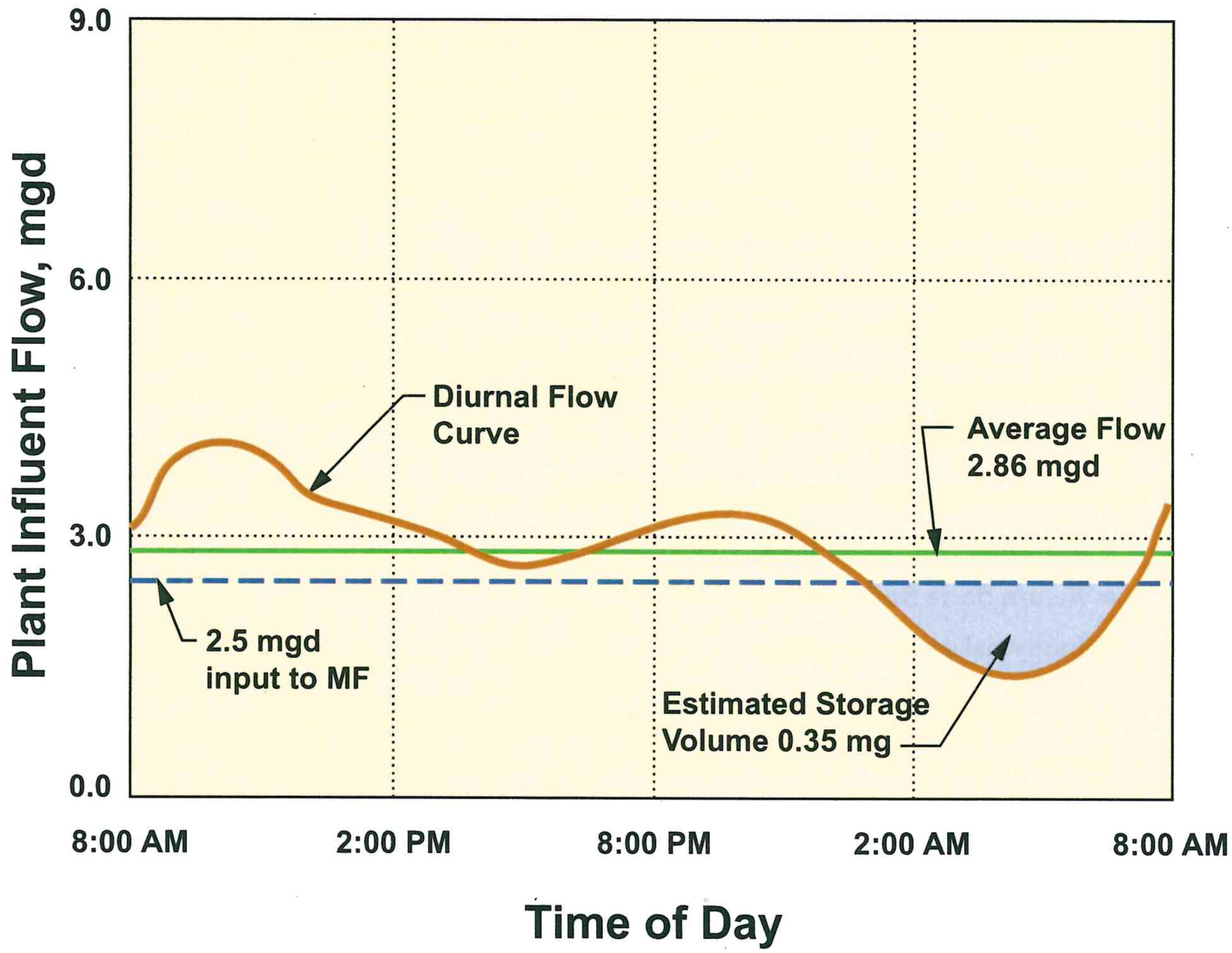

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Figure 4-3
Diurnal Storage Volume Required for 2.5 mgd Flow

Figure 4-4 shows the location of the MPBs within the plant site and the main components of the new wet weather flow improvements.

Wet weather flows are split in the new relief sewer at Control Structure No. 1, which is located at the intersection of East 5th and East "G" Streets. Part of the wet weather flow is routed via an existing 18-in sewer line to the existing headworks for complete treatment. The excess flow is routed to the new wet weather screening structure and into the MPBs.

During a rare, high-flow storm event, excess flow (i.e., greater than 12 mgd) is stored in the MPBs. The Storm Flow Return Pumps can return up to 6 mgd of flow from storage back to the primary treatment system, which has a design flow of 18 mgd (12 mgd from the headworks and 6 mgd return flow from the MPBs). If the MPBs that are on line are full, flow to them in excess of 6 mgd will overflow from MPB No. 2 into a pipeline that conveys the flow to the effluent channel of the RBC clarifiers and then onto chlorination. The peak flow rate in the overflow is 6 mgd.

The MPB's have a combined detention time of approximately 4 hours for the 20-year, peak wet weather event.

In addition to providing wet weather storage capacity, the MPBs are used for other purposes including:

- Construction and maintenance shutdowns.
- Return of plant stormwater collection.
- Return flows from seeding the RBCs.
- Industrial sewer diversions.

4.3 Description of Flow Equalization Alternatives

Based on the need to provide a steady flow of secondary effluent of 2.5 mgd to the micro-filtration system, CDM has developed three alternatives to equalize the variable plant flow, as follows:

- Alternative No. 1 - Equalize flow by storing primary effluent in the MPBs
- Alternative No. 2 - Equalize flow by storing secondary effluent in the MPBs
- Alternative No. 3 - Equalizing flow by storing secondary effluent in a new storage facility

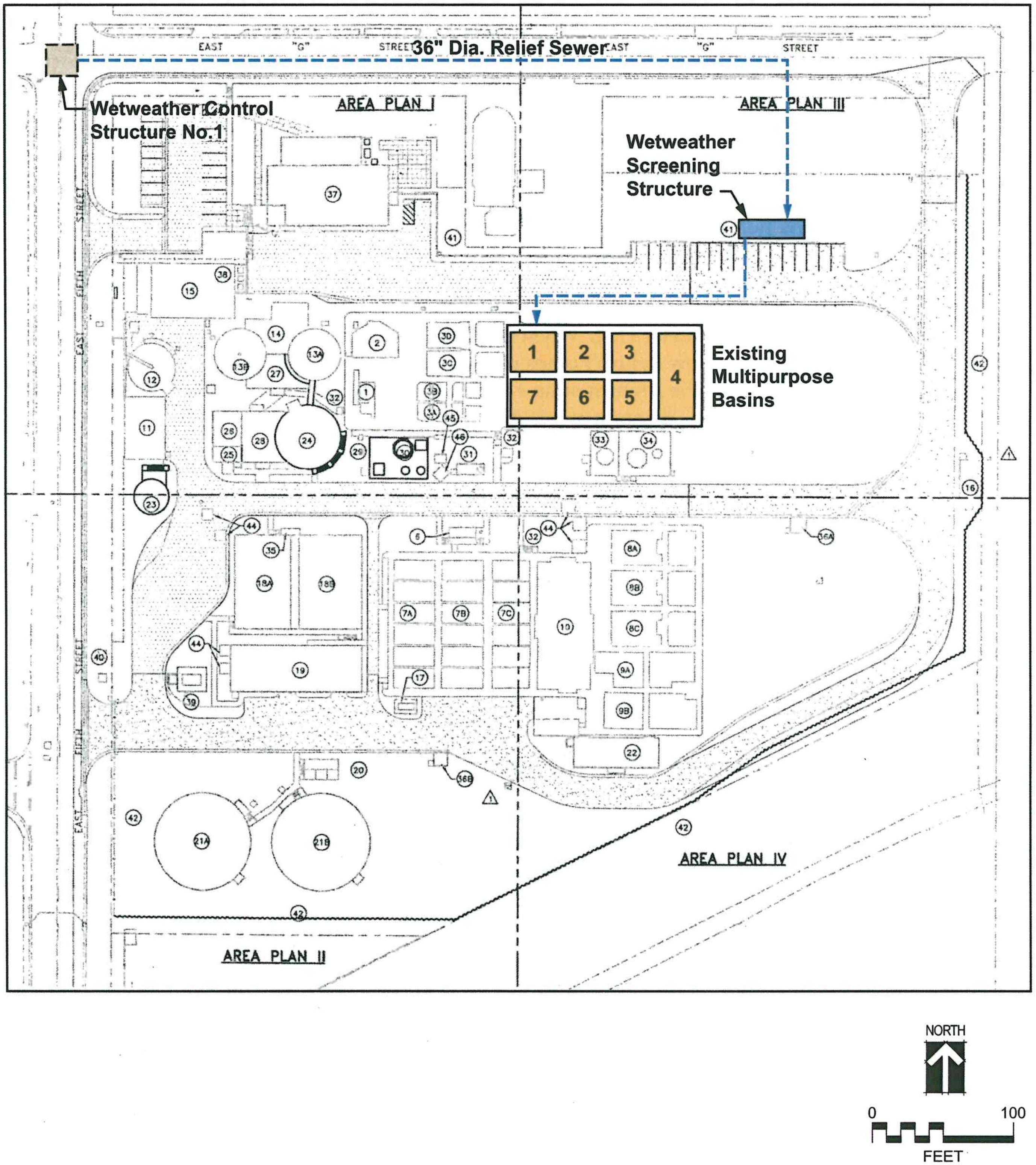

Figure 4-4

Existing Site Plan of Multipurpose Basins and Wetweather Diversion Facilities

CDM

A secondary effluent (SE) splitter box is required for all of the three alternatives. This structure will prevent chlorinated secondary effluent from flowing back into the wet well of the new MF feed pumps, which are also required under all three alternatives.

4.3.1 Alternative No. 1 - Equalizing Flow by Storing Primary Effluent Storage in MPBs

Primary effluent (PE) storage is achieved by storing flow in excess of average flow in the MPBs and pumping back the stored PE when plant influent flow drops below average flow. Flow to the MPBs is controlled by three motorized diversion valves. These valves are modulated based on maintaining a fixed rate of flow from the primary treatment system. The plant operator manually selects the PE flow rate set point. PE flow is measured by flow meter FE-2. When the PE flow rate falls below the set point, the existing Diurnal Flow Return Pumps return PE, stored in the MPBs, back to the primary effluent channel. The 2 return pumps are located in the Diurnal Flow Box. Figure 4-5 contains a schematic diagram of this Alternative No. 1.

Equalizing primary effluent flow to the secondary system will have a side benefit of providing a stable flow to the secondary treatment system and promoting secondary treatment consistency, particularly with respect to nitrification. Hence, although the Water Reuse Treatment Plant requires only 2.5 mgd influent flow, it is appropriate to equalize the entire flow for achieving a stable nitrification system and to insure that there will be a continuous flow of secondary effluent for the Number Three Water Supply System (3W).

Multi-purpose Basins 1, 2, 3, and 4 would be used for storing up to 0.62 million gallons for equalization. (Basins 1 through 3 have combined capacity of 0.38 mg.) Each of these basins will be covered and provided with aerators to keep the primary effluent from going septic. There is an existing aerator in Basin 1 that is operable. New aerators will be provided for the other basins.

The PE flow equalization is already set up. It is not anticipated that the RBC system will be needed during the dry weather flow conditions.

When a rare, high flow storm event occurs, the plant operator would change the set point on the controllers of the three modulating valves to allow the peak wet weather flow to the secondary system. Therefore, under those higher flow conditions, storage would not be needed for the MF system. However, wet weather storage would be required and this water would flow as described above in Paragraph 4.2.

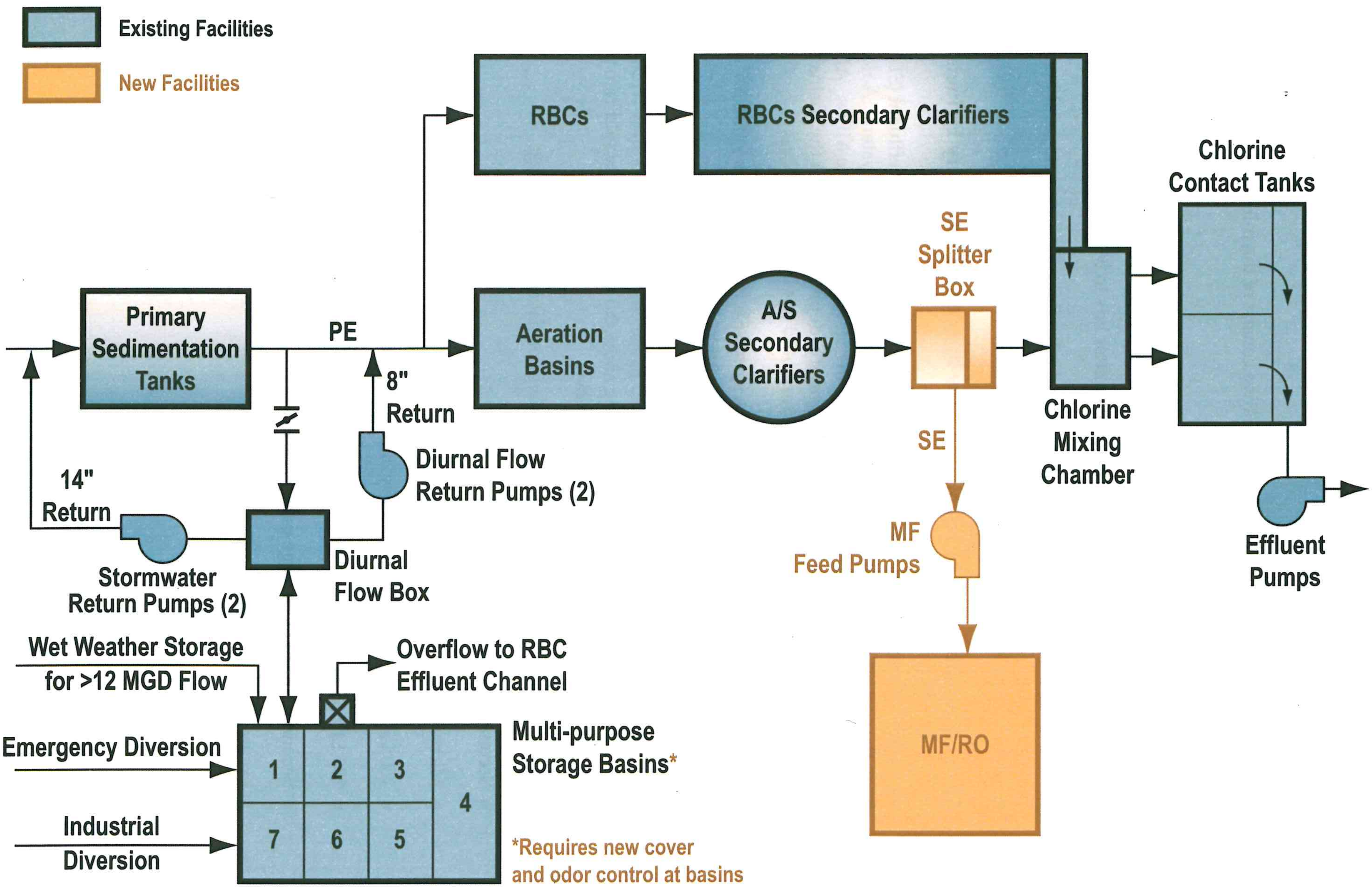

Figure 4-5
Alternative No.1: PE Storage in Existing Multi-purpose Basins

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Facilities required to implement this alternative consist of the following:

- Secondary effluent splitter box
- Microfiltration feed pumps (2)
- 70 LF 18" piping from secondary effluent splitter box to MF
- Odor control cover on MPBs 1, 2, 3 and 4 and odor duct connection to scrubbers
- New floating aerators/mixers at MPB 2, 3, and 4

4.3.2 Alternative No. 2 - Equalizing Flow by Storing Secondary Effluent in MPBs

Secondary effluent storage could be achieved by diverting secondary effluent (SE) flows from the secondary clarifiers to the MPBs and pumping it to the MF system. A new SE pump system is needed to send the flow from the SE splitter box to the MPBs. Although some flow could be routed to the MPBs by gravity, depth of storage in the basins would become limiting and adequate storage volume could not be provided.

Another set of pumps is needed to pump the stored SE from the MPBs to the MF system. Figure 4-6 presents a schematic diagram of this alternative.

Multi-purpose Basins 3, 4, and 5 will need to be used for storing up to 0.5 million gallons for equalization. New aerators will be needed in the basins to keep the flow from going septic. Remaining MPBs 1, 2, 6, and 7, having about 0.5 mg capacity, would remain for emergency storage and maintenance purposes.

Facilities required to implement this alternative consist of the following:

- Secondary effluent splitter box.
- Microfiltration feed pumps (2)
- 260 LF 18" piping from secondary effluent splitter to storage basins
- 180 LF 18" piping from storage basin to MF
- Secondary effluent storage pumps (2)
- Modification to MPBs for MF feed pumps

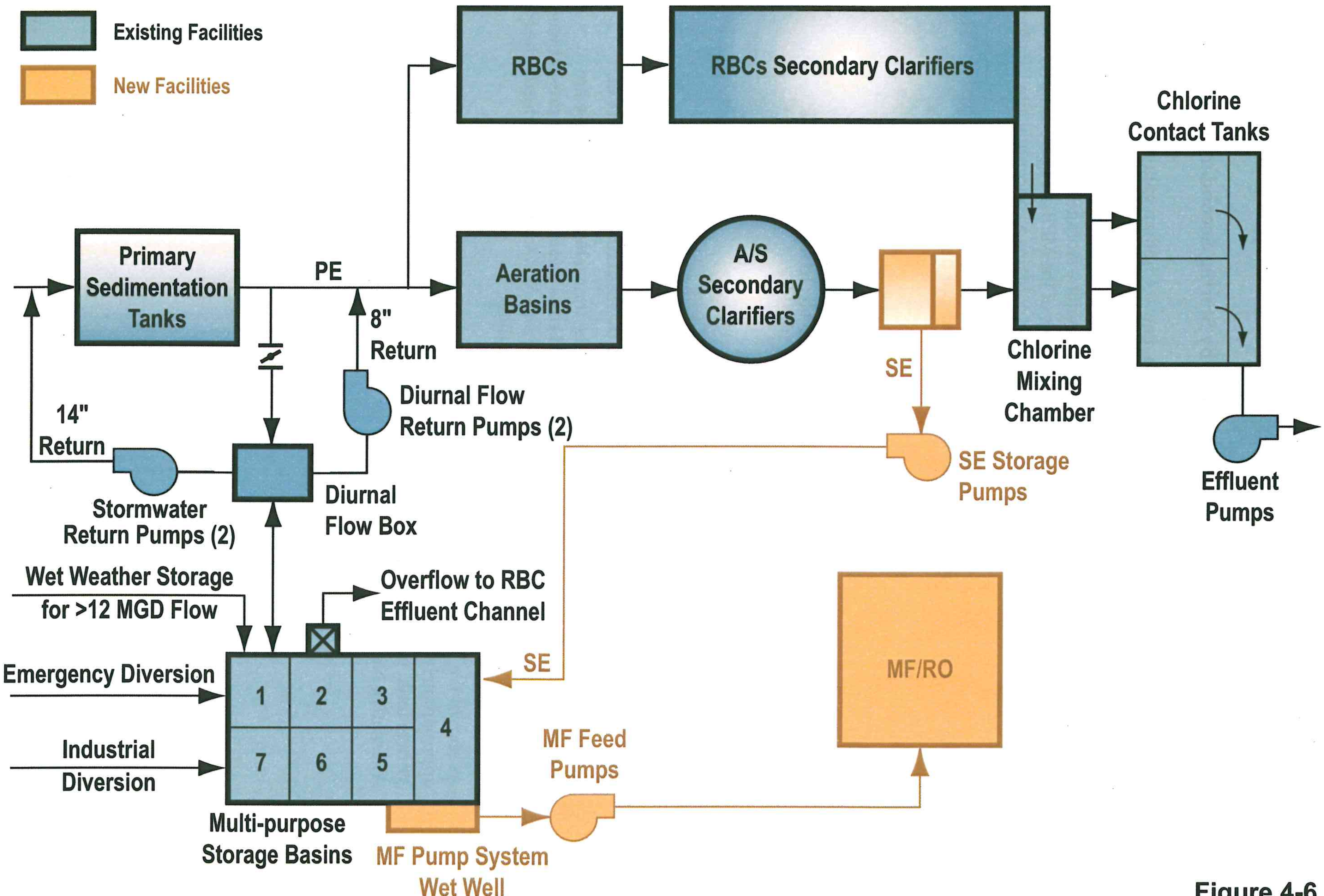

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Figure 4-6

Alternative No.2: SE Storage in Existing Multi-purpose Basins

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4.3.3 Alternative No. 3 - Equalizing Flow by Storing Secondary Effluent in a New Storage Facility

In lieu of storing secondary effluent in the MPBs, new 0.4 million gallon storage tank would be provided to store secondary effluent going to the MF system. Pumps would be required to pump flow from the SE splitter box to the new above grade storage tank. Another set of pumps would be needed to pump from the storage tank to the MF system. This alternative avoids any impacts to the operations of the MPBs. Figure 4-7 presents a schematic diagram of this alternative, and Figure 4-8 contains a conceptual site location for this tank.

Facilities required to implement this alternative consist of the following:

- Secondary effluent splitter box
- MF feed pumps (2)
- 200 LF 18"piping from secondary effluent splitter box to new storage tank
- 200 LF 18" piping from storage tank to MF
- Secondary effluent storage pumps (2)
- 0.4 million gallon above ground concrete storage tank with pile foundation

4.4 Economic Evaluation of Flow Equalization Alternatives

Construction and O&M costs were estimated for the three alternatives. They are summarized in Table 4.1. Estimated annual O&M costs were converted to present worth values. Elements common to all alternatives were removed to show more clearly the cost and PW value differences among the three alternatives.

As shown the Table 4-1, the estimated present worth of Alternative No. 2 is approximately 30% less than Alternative No. 1. Alternative No. 3, construction a new storage tank, has the highest present worth value and is nearly three times higher than Alternative No. 2.

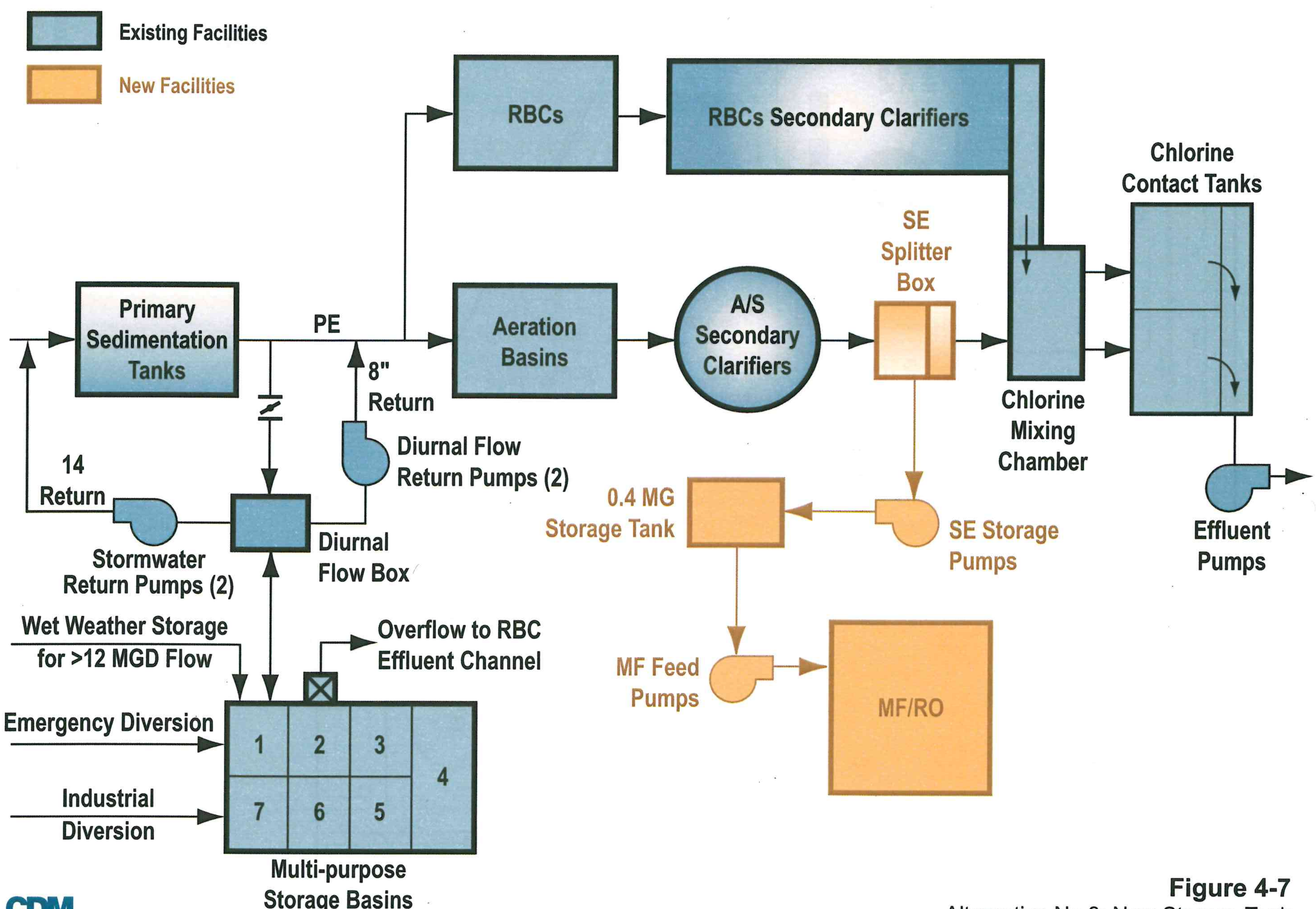

Figure 4-7
Alternative No.3: New Storage Tank

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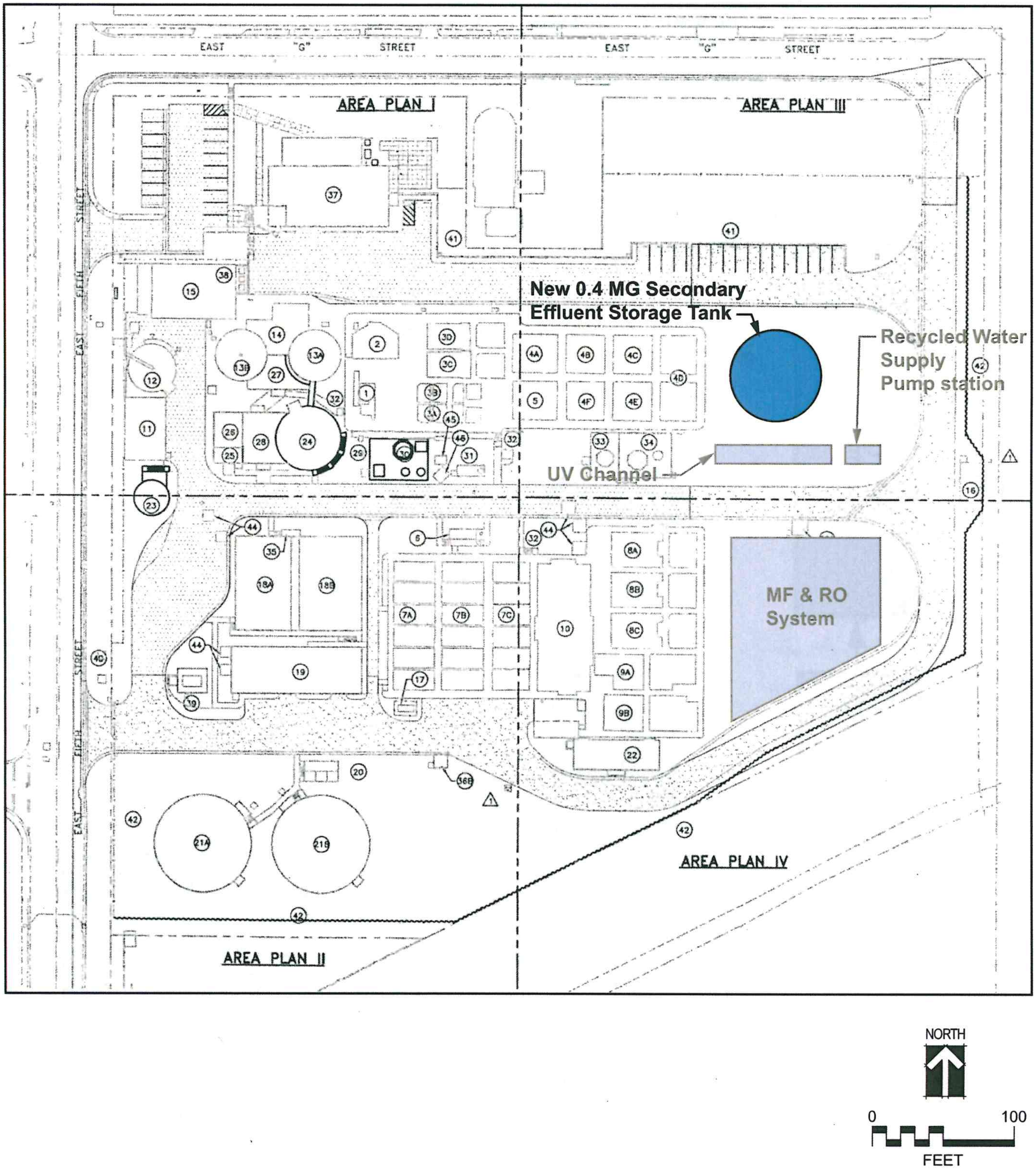

Figure 4-8
Conceptual Site Plan for New 0.4 MG Storage Tank at Benicia

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Table 4-1
Summary of Economic Analysis of Flow Equalization Alternatives

Components	***Unit Price***	***Alternative No. 1 - Equalize Primary Effluent Flow***	***Alternative No. 2 - Equalize Secondary Effluent (SE) Flow***	***Alternative No. 3 - Construct New SE Storage Tank***
Secondary Effluent Splitter Box (8x6x10'D) (1)	$25,000 LS	$25,000	$25,000	$25,000
Microfiltration Feed Pumps (2.5mgd, 20 psi, 30 hp) (1)	$17,000 ea	$34,000	$34,000	$34,000
18" PVC Pipe	$50 / LF	$4,000	$22,000	$20,000
Odor Control Multi-Purpose Basin Cover (35x30 span per basin)	$50/ SF	$180,000	$0	$0
Foul Air Duct	$40 /LF	$11,000	$0	$0
Floating Aerators (5 hp)	$7,000 ea	$28,000	$0	$0
Secondary Effluent Storage Pumps (2.5 mgd, 6 psi, 10 hp, w/VFD))	$15,000 ea	$0	$30,000	$30,000
0.4 MG Above Ground Storage Tank (74' Dia x 16' SWD)	$0.75 / gal	$0	$0	$300,000
Tank Pile Foundation	$3000/ pile	$0	$0	$105,000
Modification to Multi-Purpose Basin	$75,000 LS	$0	$75,000	$0
Subtotals		$282,000	$186,000	$514,000
Add 50% of Mech Equip Cost for Electrical/I&C		$31,000	$32,000	$32,000
Subtotals		$313,000	$218,000	$546,000
Add 25% Contingency		$78,000	$55,000	$137,000
Subtotals		$391,000	$273,000	$683,000
Add 15% Contractor OH & Profit		$59,000	$41,000	$102,000
Total Estimated Construction Costs		**$450,000**	**$314,000**	**$785,000**
Delete Estimated Costs of Common Elements		$109,000	$109,000	$109,000
Total Est. Construction Cost, Excluding Common Elements		***$341,000***	***$205,000***	***$676,000***
Add 35% for Engineering & CM		$119,000	$72,000	$237,000
Total Est. Capital Cost, Excluding Common Elements		**$460,000**	**$277,000**	**$913,000**
Estimated Annual O&M Power Cost, excluding common elements				
Estimated Power Unit Cost, $s/kWHr	$0.12			
Diurnal Flow Return Pumps ($/yr)		$4,027	$0	$0
Floating Aerators ($/yr)		$2,013	$0	$0
Secondary Effluent Storage Pumps ($/yr)		$0	$8,054	$8,054
Total Estimated Annual Power Costs		$6,040	$8,054	$8,054
Estimated Present Worth of Power Costs, excluding common elements		**$69,000**	**$92,000**	**$92,000**
Total Est. Capitalized Cost (rounded), Excluding Common Elements		**$529,000**	**$369,000**	**$1,005,000**

(1) Elements common to all alternatives

4.5 Discussion and Comparison of Flow Equalization Alternatives

Table 4-2 contains a summary of the advantages and disadvantages of each of the alternatives.

4.6 Recommended Flow Equalization Alternative

Based on the discussion of the advantages and disadvantages of each of the alternatives and the results of the economic analysis, CDM recommends that the City proceed with implementation of Alternative No. 2 – Equalizing Flow by Storing Secondary Effluent in the MPBs.

Table 4-2
Summary of Flow Equalization Alternatives

	Alternative No. 1 Multi-Purpose Basins for Equalizing Primary Effluent	**Alternative No. 2 Multi-Purpose Basins for Equalizing Secondary Effluent**	**Alternative No. 3 New Equalization Basins**
Dry Weather Operational Impacts	Multi-purpose Basins (MPB) are already piped to accept primary effluent for equalization and there are existing Diurnal Flow Return Pumps in place to return effluent to the primary effluent channel. Can use Basins 1, 2, 3, 4 (volume approx 0.625 million gallons) to equalize primary effluent flow to secondary treatment system and improve secondary treatment stability. Will need to add aluminum covers on the basins to reduce odors from primary effluent. Covered basins will be more difficult to clean and maintain than uncovered basin. Primary effluent solids that settle in the basins will need to be removed regularly or agitated by mixer to prevent septicity. New mixers will be needed in Basins 2, 3, and 4.	Can use Basins 3, 4, and 5 (volume approx. 0.5 million gallons) for secondary effluent storage Basins 1, 2, 6, and 7 (0.5 million gallons total) can be retained for primary effluent equalization or primary effluent storage during maintenance events.	No dry weather operation impacts Full use of multi-purpose basins for plant operations and maintenance as designed.
Emergency and Maintenance Impacts	Use for Emergency Plant Diversion (Basin 1) may be restricted if there is already primary effluent in the basin. Maintenance will be more difficult with covers over basins	Emergency and maintenance use would be limited to Basins 1, 2, 6, and 7	No impact to Emergency Plant Diversions or maintenance.
Wet Weather Operational Impacts	During wet weather event, flows in excess of 12 MGD to the plant will be diverted to the MPBs from Relief Sewer Control Structure No. 1. Prior to using the MPB for wet weather flow storage (at some point before plant flow is 12 MGD), primary effluent stored in the basins will have to be drained from the basins. Once wet weather event subsides, and when storage is no longer needed, the MPBs will need to be drained of the storm flow and cleaned prior to returning it for use primary effluent storage.	During storm, flows in excess of 12 MGD to the plant will be diverted to the MPBs from Relief Sewer Control Structure No. 1. Prior to using the MPB for wet weather flow storage (at some point before plant flow is 12 MGD), secondary effluent stored in the basins will have to be drained. Once wet weather event subsides, and when wet weather storage is no longer needed, the MPBs will need to be drained of the storm flow and cleaned thoroughly prior to returning it for secondary effluent storage. Since secondary effluent stored will go directly to the MF system, the MPBs will need to be cleaned thoroughly.	No wet weather operational impacts Full use of basins for all storm or emergency events

5. Electrical Power Supply

On December 22, 2004, CDM's electrical engineer made a site visit to the City's WWTP to gain an understanding of electrical power supply and distribution, standby power capacity and operations and potential future loads. Plant operations staff discussed limitations, system characteristics and concerns with CDM. The discussions and preliminary conclusions presented below are based on this initial site visit and the information contained in the construction drawings of the 1998 Improvement Project. Additional field investigation will be required to confirm that the record drawings accurately represent the existing switchgear capabilities during development of electrical service preliminary design.

5.1 Existing PG&E Electrical Service

Electrical power is provided to the existing Benicia WWTP from the PG&E overhead pole line which runs along 5th street at the west side of the treatment plant property. An underground conduit extends the PG&E primary cables from the riser pole to the single PG&E transformer located adjacent to the Blower Building. The transformer provides power at 480/277 VAC to the Main Switchboard located inside the Blower Building. A review of the record drawings for the Wastewater Treatment Plant Improvement Project (dated May 2001) and a preliminary site inspection indicate that the Main Switchboard is rated for a maximum of 4,000 amps, has two spare breakers (400 A and 800 A) and several spaces available for future expansion.

The PG&E pole line in East "G" Street, along the northern plant property line appears to be only a single phase circuit serving residential customers on the adjacent street. This line presently extends only part of the distance to the plants eastern property line.

5.2 On-Site Emergency Electric Power Generation

In approximately the spring of 2002, a nominally rated 1,000 kW 480/277 VAC standby power generator set was furnished and installed near the Blower Building main switchboard. This natural gas powered generator installation includes a small on-site propane storage tank, which can provide approximately 12 hours of operation, if the natural gas supply is interrupted. City staff advised that the new generator was designed to operate several hours a day when required to remove the entire plant load from the PG&E service.

The PG&E service transformer and the standby generator are connected to the plant's main automatic transfer switch (located within the Main Switchboard) in a manner that allows any load within the plant to be connected to the electrical distribution system. They thus provide dual sources of power to all components. The standby generator (1,000 kW) is connected to the emergency source side of the plant's automatic transfer switch. Preliminary discussions with plant staff indicate that the 1,000 kW generator has adequate capacity to allow full normal operation of the plant. The generator's 1,000 kW rating is equivalent to approximately 1,500 amps at 480 volts. Since the main switchboard has a full load capacity of 4,000 amps, the switchboard can supply more load through the normal PG&E transformer than can be served during a PG&E outage by the 1,000 kW generator. Additional field investigation is required to determine the typical loading of the 1,000 kW generator during PG&E power outages.

5.3 History of Utility Power Outages

PG&E supplied CDM with its records of power outages over the last three years. PG&E reports five "momentary interruptions" and six sustained interruptions. Five of the sustained interruptions ranged between 50 and 80 minutes. One lasted nearly seven hours. Although the data set it too small for statistical analysis, still it appears that PG&E power supply to the WWTP has been highly reliable. However, past history is no guarantee of future reliability, particularly given the reported tenuousness of power supply reliability in California. Also, future power outages are likely to be summertime "brownouts" when other sources of water are scarce or not available.

Therefore, this conceptual design is based on providing full, standby power for the Water Reuse project in order to sustain a continuous supply of 2.0 mgd 100 percent of the time.

5.4 Existing Plant Power Demands

City records indicate that the maximum electrical demand was 520 kW occurring in December 2003. This maximum demand has remained relatively constant over the period 2002 through 2004 (with a low maximum demand of 420 kW occurring in June 2004).

Other than the power requirements of the proposed Water Reuse Project, it is anticipated that additional future demands on the plant's electrical service will be relatively low due to site constraint limitations.

5.5 Estimated Electrical Power Demands for the New Water Reuse Project

Electrical power demands were made based on both preliminary process equipment selection and sizing, and the location of the process units. Table 5-1 contains a summary of estimated electrical power demands for the three alternatives presented in Section 2. This indicates that the new water reclamation facility under Alternative No. 1 would add approximately 960 additional horsepower of connected load to the existing plant. Although the actual demand load will be less (preliminary calculations indicate a normal operational load of approximately 540 horsepower would be anticipated), the connected load has been used to provide a conservative basis of analyzing the electrical service capacity at the Benicia WWTP.

Table 5-1 Summary of Electrical Power Demands for Alternative Water Reuse Treatment Sites						
	Alternative No. 1		Alternative No. 2		Alternative NO. 3	
	MF/RO/UV @ Benicia		MF/UV @ Benicia; RO @ Valero		MF @ Benicia; RO/UV @ Valero	
	Connected	Demand	Connected	Demand	Connected	Demand
Estimated Load @ Benicia, HP	960	540	510	250	480	240
Estimated Load @ Benicia, kW	720	410	380	190	360	180
Estimated Load @ Valero, HP	0	0	520	330	580	370
Estimated Load @ Valero, kW	0	0	390	250	440	270
Totals, HP	***960***	***540***	***1,030***	***580***	***1,060***	***610***
Totals, kW	***720***	***410***	***770***	***440***	***800***	***450***

As discussed above, all process demand loads should be provided with standby power in case the PG&E source fails. The loads that must be served during a utility power outage will be the determining factor in the size of a standby power generator. If the standby power load exceeds the available excess capacity of the plant existing 1,000 kW generator, a new generator would be required to serve the process facilities whether at Benicia or Valero.

5.6 New Water Reuse Project Electrical Service Alternatives

The two alternatives for supplying power to the new reclamation plant at Benicia are as follows:

- Alternative No. 1: Provide power through the existing plant service
- Alternative No. 2: Provide power thought a new service dedicated to the Reuse Plant.

Each of these alternatives has advantages and disadvantages associated with implementation. Even if the Alternative No. 2 (separate reclamation service) is selected, it is anticipated that there will need to be some additional loads (aeration blowers) added to the existing plant service to accommodate the process modification to nitrification. A preliminary estimate of the magnitude of these loads indicates that they will total approximately 75 horsepower.

A preliminary calculation indicates that the existing 4,000 amp electrical service has adequate capacity to supply both the existing plant (approximately 920 connected load amps, including the 25% contingency/overload factor as required by the National Electrical Code) and the Water Reuse Facility (approximately 1,825 connected load amps also including a similar NEC 25% factor). Based on these conservative calculations, there are approximately 450 amps (approximately 365 horsepower) of future expansion capacity through the existing service after the reuse facility is added.

5.6.1 Alternative 1 – Water Reuse Treatment Plant served via the exiting WWTP service:

Advantages:

- May prove more cost effective than providing a new (second) electrical service. (Awaiting cost information from PG&E.)
- Minimizes schedule and cost issues (e.g. no need to coordinate with PG&E)
- The existing standby generator possibly could serve the Water Reuse Treatment Plant equipment.

Disadvantages:

- Underground electrical service conduits will need to be routed through a physically congested area of the plant.
- Utilizes a significant portion of the existing WWTP service thereby reducing the future WWTP expansion capabilities without significant electrical upgrades.

5.6.2 Alternative 2 – New service dedicated to the Water Reuse Treatment Plant:

Advantages:

- Provides complete separation of the power metering (2 separate PG&E bills) to the Water Reuse Treatment Plant.
- More flexibility for energy metering and potential energy wheeling contracts between PG&E and the refinery.
- Potential for easier installation (smaller underground conduit and potential for entering the site from the north property line as opposed to connection of the new feeder circuit breaker, metering devices and cables into the existing plant main switchboard and routing across the plant site).
- Less impact on the electrical system design if the RO component of the reclamation process equipment is located at the refinery.
- Potentially more flexible for utilizing alternative energy sources.

Disadvantages:

- Requires convincing PG&E that a new (second) service is required at this location which already has an existing service with adequate capacity.
- May be higher in capital cost, depending on PG&E estimate.

- Potential design scheduling issues to coordinate with PG&E.

5.7 Required Additional Investigations

Before a final electrical service recommendation can be made, the following issues require additional investigation and resolution:

- Explore the viability and costs associated with a second PG&E service located on the northeast corner of the WWTP property and dedicated to the Water Reuse Plant. (PG&E was contacted on 1/19/05 and the electrical service needs of the Reuse Project were outlined. Two basic questions were asked; (1) can a second service be installed, and (2) what would be the costs associated with this installation. PG&E has not yet provided adequate information to incorporate into this Draft TM.)

- Confirm the assumption of minimal future electrical loads (in addition to the Water Reuse Project) at the plant site.

- Refine the physical location of the Reuse Project electrical loads. The electrical service recommendations presently incorporated in this TM are based on the most conservative loading conditions, that is with the entire process load is located at the treatment plant (versus the RO equipment being located at the refinery site.)

5.8 Analysis of Electrical Supply Alternatives from a Siting Perspective

5.8.1 Electrical Power Supply at Benicia WWTP

As noted above, there are basically two electrical supply alternatives at the Benicia WWTP, namely:

- Alternative No. 1 – Electrical Service through the existing PG&E service to the WWTP

- Alternative No. 2 – A new electrical service from PG&E dedicated to the propose WRTP

In addition to the primary power supply of these two alternatives, there are additional components and capacity issues that need to be evaluated and that impact both the economics of the two alternatives. These are:

- Standby power for back up to PG&E

- The capacity of both the primary services and standby supplies relating to the different load capacities, based on the three siting alternatives for the locations of the treatment process units, as detailed in Sections 2 and 3.

Based on the preliminary analysis stated above, the existing PG&E service and plant Main Switchboard components could marginally serve existing demands plus the estimated additional loads from the Water Reuse Project under existing Alternative No. 1, MF/RO/UV located at the Benicia WWTP plus the estimated additional future loads for the existing WWTP. Also, analysis of the capacity of the 1,000 kW standby generator finds that its capacity would be totally consumed or possibly exceeded, if it were called to power both systems. Based on the preliminary nature of the equipment selection at this stage in the project development, it appears that a new generator would be required for the Water Reuse Project electrical loads.

However, under siting Alternative Nos. 2 and 3, which are based on locating the RO and the RO and UV, respectively, at Valero, the estimated new electrical demands at the Benicia WWTP are estimated to be approximately 230kW less. Thus, for siting Alternative Nos. 2 and 3, the electrical power demands of the Water Reuse Project facilities at the Benicia WWTP would have less impact on the existing service. Also, there is a greater likelihood that the existing standby generator would be able to accommodate this additional load.

Hence, a matrix of alternatives, components and estimated costs have been developed to aid in the decision making process and is presented in Table 5-2. As shown in the table under Alternative No. 1, a new circuit breaker would be installed in the existing plant Main Switchboard in the Blower Building and approximately 600 feet of buried electrical duct bank for conduit and cable from the existing switch gear to the new MF/RO building to provide electrical supply to the Water Reuse Treatment Plant.

For both alternatives, there appears to be a significant cost difference to locating the RO System at Valero.

5.8.2 Electrical Power Supply at Valero Refinery

Power supply in the vicinity of the location proposed for the Water Reuse Treatment facilities (please refer to Figures 2.5 and 2.6), is served from Valero Substation C (Sub C), which feeds Substation 26 (Sub 26). Both substations are at 4,160 volts. The existing Valero Wastewater Diversion Area (WDA) is served from Sub 26, which is located near the NW corner of the WDA. Sub 26 does not have capacity to serve the requirements of siting Alternative Nos. 2 or 3. However, the buried cable between Sub C and Sub 26 has capacity for increased power up to 750 kVA. One option would be replace Sub 26 with a new 750 kVA transformer and connect both the existing and new (Water Reuse system) through new switchgear. However, approximately 350 feet of new buried cabling would be required from the new Sub 26 back southerly to the Water Reuse Treatment facilities.

Another option would be to tie into Sub C and run new cabling to a point nearer the proposed location of the Water Reuse Facilities and set a new 750 kVA transformer there. Also included in the option would be to splice in, in series, the existing Sub 26 to the north. This series cabling would be required because there are not spare contactors in Sub C. Hence, the contactors that now serve Sub 26 would have to serve both Sub 26 and the new substation/transformer for the Water Reuse Project.

Table 5-2 ***Conceptual Cost Estimates Of Electric Power Supply Alternatives***			
	Alt. No. 1 – MF/RO/UV @ Benicia	**Alt. No. 2 – MF/UV @ Benicia; RO @ Valero**	**Alt. No. 3 – MF @ Benicia; RO/UV @ Valero**
	Estimated Construction Cost	Estimated Construction Cost	Estimated Construction Cost
	$1,000's	$1,000's	$1,000's
New Feeder circuit breaker in existing switchgear and Cabling to Reuse Plant at Benicia	$90	$70	$70
500 kW Standby Generator at Benicia	$100	Not required	Not required
New 500 kW Standby Generator at Valero	Not applicable	$100	$100
New 4,160 kVA cable, transformer, and switchgear at Valero	Not applicable	$90	$90
Subtotals	$190	$260	$260
Contingency @ 25%	$50	$70	$70
Subtotals	$240	$330	$330
Contractor OH&P at 15%	$40	$50	$50
Total Estimated Construction Costs	**$280**	**$380**	**$380**

Notes:

1. Only the differential costs of the alternatives are included in this estimate. The costs for the electrical system (480 volts and below) at the Water Reuse Treatment Facilities (whether located at Benicia or Valero) are not included in the above table, since they will be comparable down stream of the electrical supply source.

Detailed review of record drawings and field investigations would be needed to determine the best, most cost-effective modifications and additions required to serve project components. For this analysis, it has been assumed that power would be obtained from Sub C, new buried cabling would be provided to a location next to the Water Reuse Treatment Facilities and a series connection would be made back to Sub 26. The new transformer would be sized at 750 kVA, although further investigations might find that a 500 kVA unit would suffice.

Based on the above, conceptual estimates were made of the cost to connect to Sub C, add cable to the new Water Reuse area and add a 750 kVA transformer and switchgear. Cost of motor control centers and other electrical services were considered the same, where the facilities are located at Benicia or at Valero.

In keeping with the criterion of providing 100 % standby power supply (please see Paragraph 5.3, above), standby generators have been included in the necessary electrical power supply components and related costs for the siting Alternative No. 2 and 3..

6. Alternative Energy Sources

6.1 Overview of Alternative Energy Sources

Alternative energy sources are resources that are renewable and are usually less polluting than those derived from the burning of fossil fuels. Alternative energy sources include: biomass, geothermal, hydroelectric, solar, wind and ocean.

- **Biomass:** Biomass is renewable energy that is produced from organic matter. Biomass fuels include: wood and forest and mill residues, animal waste, grains, agricultural crops, aquatic plants and organic sludge from wastewater treatment plants. These materials are used as fuel to heat water for steam or are processed into liquids and gases, which can be burned to do the same thing. It is estimated (per PG&E web site) that biomass will have the largest increase among renewable energy sources, rising by 80 percent and reaching 65.7 billion kWhr in 2020. The City's WWTP stabilizes the biomass removed from the process by anaerobic digestion. This process converts organic material to methane gas, which is used in a boiler to heat the biomass (sludge) to sustain the process.

  The use of digester gas as an alternative renewable fuel source is discussed below.

- **Geothermal:** Geothermal energy uses heat from within the earth. Wells are drilled into geothermal reservoirs to bring the hot water or steam to the surface. The steam then drives a turbine-generator to generate electricity in geothermal plants. In some places this heat is used directly to heat homes and greenhouses, or to provide process heat for businesses or industries. The City of Reykjavik, Iceland is heated by geothermal energy. Most geothermal resources are concentrated in the western part of the United States. Geothermal heat pumps use shallow ground energy to heat and cool homes almost anywhere. With technological improvements much more power could be generated from hydrothermal resources. Scientists have been experimenting by pumping water into hot, dry rock generally located 3-6 miles below the earth's surface for use in geothermal power plants.

  The City of Santa Rosa supplies an average of 11 mgd of recycled water to the Geysers in the Mayacamas Mountains, northeast of Santa Rosa in Sonoma County. The recycled water is injected into the steam fields for sustaining steam production and electrical generation.

- **Hydroelectric:** Hydroelectric energy employs the force of falling water to drive turbine-generators in order to produce electricity. Hydropower currently produces more electricity than any other alternative energy source. Development of any significant additional hydroelectric power in the U.S. is unlikely, given concerns about potential adverse impact that large-scale hydroelectric facilities may have on the environment.

- **Solar:** Solar energy is generated without a turbine or electromagnet. Special panels of photovoltaic cells capture light from the sun and convert it directly into electricity. The electricity is stored in a battery. Solar energy can also be used to directly heat water for domestic use (solar thermal technology). According to PG&E, the domestic photovoltaic (PV) industry could provide up to 15% of new U.S. peak electricity capacity that is expected to be required in 2020. Solar PV systems are discussed further below.

- **Wind:** Wind energy can also be used to produce electricity. As wind passes through the blades of a windmill, the blades spin. The shaft that is attached to the blades turns and powers a pump or turns a generator to produce electricity. Electricity is then stored in batteries. The speed of the wind and the size of the blades determine how much energy can be produced. Wind energy is more efficient in windier parts of the country. Most wind power is produced from wind farms—large groups of turbines located in consistently windy locations, such as the Altamont Pass in the Bay Area. Wind, used as a fuel, is free and non-polluting and produces no emissions or chemical wastes. Although wind-powered electricity is gaining in popularity in some locales, visual impacts and impact on raptors and other birds are of concern. Wind turbine systems are discussed further below.

- **Oceans:** Oceans, which cover more than 70% of the Earth, contain both thermal energy from the sun's heat and mechanical energy from tides and waves. Ocean thermal energy conversion (OTEC) converts solar radiation to electric power. OTEC power plants use the difference in temperature between warm surface waters heated by the sun and colder waters found at ocean depths to generate electricity. The power of tides can be harnessed to produce electricity. Tidal energy works from the power of changing tides but it needs large tidal differences. The tidal process utilizes the natural motion of the tides to fill reservoirs, which are then slowly discharged through electricity-producing turbines. Wave energy conversion extracts energy from surface waves, from pressure fluctuations below the water surface, or from the full wave. Wave energy uses the interaction of winds with the ocean surface. This technology is still in the exploratory phases in the United States and is being investigated for large capacity (mega-Watt) systems. Hence, energy recovery from the oceans is not feasible for the Water Reuse Project.

6.2 Rebate Programs for Alternative Energy Generation

In September 2000, Assembly Bill 970 was approved, which called for the creation of more energy supply and demand programs. As a result, in March 2001, the California Public Utilities Commission (CPUC) issued a decision creating the Self-Generation Incentive Program (SGIP) to offer financial incentives to their customers who install certain types of distributed, self-generation facilities to meet all or a portion of their energy needs, up to 1.5 MW, although the maximum incentives basis remains capped at 1,000 kW.

These facilities must be certified to operate in parallel with the electric system grid (not back-up generation) and meet other criteria established by the California Public Utilities Commission. While residential customers are not barred from the program, it was designed primarily with business and large institutional customers in mind. The California Energy Commission offers a similar program that is available to customers who install renewable generation sources, such as photovoltaics and wind turbines less than 30 kW.

"Self-generation" refers to distributed generation technologies (microturbines, small gas turbines, wind turbines, photovoltaics, fuel cells and internal combustion engines) installed on the customer's side of the utility meter that provide electricity for either a portion or all of that customer's electric load. Financial incentives are provided to the targeted distributed generation technologies as summarized in Table 6-1. The CPUC is

Table 6-1
Summary of Self-Generation Program Incentive Level

1. Operating on renewable fuel
2. Operating on non-renewable fuel
3. Using sufficient waste heat recovery
4. Meeting reliability criteria

Program Incentive Category	Maximum Incentive Offered ($/watt)	Maximum Incentive % of Eligible Project Cost	Minimum System Size (kW)	Maximum System Size Incentivized (kW)	Eligible Generation Technologies
Level 1	$4.50	50%	30	1,000	• PV Solar • Fuel Cells[1] • Wind Turbines
Level 2	$2.50	40%	None	1,000	Fuel Cells[2 & 3]
Level 3 Renewable	$1.50	40%	None	1,000	• Microturbines[1] • IC engines and small gas turbines[1]
Level 3 Non-renewable	$1.00	30%	None	1,000	• Microturbines[2,3 & 4] • IC engines and small gas turbines[2,3 & 4]

currently considering proposed modifications to current incentive levels and program requirements. PG&E administers the Program in its service territories.

The CPUC authorized a statewide annual budget of $100 million through 2004, allocated equally between Levels 1, 2, and 3. Program Administrators may reallocate incentive funds to Level 1 projects, according to market demand. Level 1 or Level 2 allocations may not be transferred to Level 3-N projects without CPUC approval. Program Administrators may also use administrative funds to pay incentives, if such funds are not required for their original purpose.

6.3 Potentially Applicable Alternative Energy Sources for the Water Reuse Project

As mentioned previously, this project will require an additional 750 kW (connected load) of reliable power supply to the City's WWTP. Generally, for a project of this type and size of demand, power would be supplied by PG&E from the grid. Some alternative sources may be found economically feasible to supplement utility supplied power. Several of the above mentioned sources are intuitively not practical as additional energy sources for the City's proposed Project. Potentially feasible supplemental sources include PV solar and wind.

6.3.1 Photovoltaic Solar Systems

Photovoltaic (PV) Solar Systems can be designed either as "Off-Grid" systems or "On-Grid" systems. Naturally, both systems can generate power only when the sun is shining. Also, their production varies throughout the day, owing to daily and seasonal variations in sunlight and intensity. Off-Grid systems can supply power only during daylight hours when they are generating electricity. Or, they can be linked with batteries and supply power around the clock. In the case of a system with a capacity of 750 kW, the cost of the batteries would be prohibitive. Hence, only an On-Grid system might be feasible. However, utility (PG&E) or other power source would need to be linked with the On-Grid system in order to provide a continuously reliable source of power.

The conceptual cost of an On-Grid system would be in the $7 to $8 per installed Watt range. This unit cost translates to approximately $5 million. Costs for an at-ground structural system to support the PV panels would be in addition to the $5 million.

Standard PV panels are constructed of modules approximately 14 sf each. Each panel can generate up to 180 Watts. However, the panels must be down-rated 30% to account for inefficiencies associated with converters and other equipment required to convert the solar energy into useable electricity. Hence, a system with an out-put capacity of 750 kW, requires approximately 83,000 sf of panels, or about 2 acres. This is an area approximately 300 ft by 300 ft.

Even though there maybe significant rebates (up to 40%) from the California Energy Commission for solar systems in this size range, still the space limitations at the City's WWTP preclude this alternative energy source from further consideration.

6.3.2 Wind Turbine Systems

Because of variability of wind speed and duration, wind turbines cannot be considered a primary source of power. Generally, for commercial and industrial installations, they are connected to the utility grid at the owner's meter. When adequate wind is available to produce electricity, the meter runs backwards. In the case of the Water Reuse Treatment System, depending on the size of the turbine and the demand of the plant, a turbine could be used to off-set some of the power demand, but on a variable basis related to wind velocity and duration. In the size range of 600kW to 1,000kW, a conceptual installed unit cost for wind turbines is approximately $800 to $1,000 per kW. Hence, for a wind turbine system that would supply 750 kW of electricity, the estimated installed additional cost would be approximately $0.9 million. (Installed unit cost increases as the size of the system decreases.) Rebate programs through the California Public Utilities Commission could reduce this cost by up to one half. In the size range stated, the height of these wind turbines is approximately 150 feet. To determine whether a wind turbine would be a cost-effective supplemental power supply for this Project, an analysis of wind velocity statistics would need to be performed along with more details on installed cost of the equipment. Owing to the high installed costs and potential impacts such as visual, raptors, and noise, wind power is likely not feasible as a supplemental power supply for the City's Water Reuse Project.

6.4 Renewable-Fueled Energy Supply Sources

The energy needs of the Water Reuse Project could also be supplemented by renewable systems that can burn digester (methane) gas. These include:

- Fuel Cells
- Micro-Turbines
- Generators driven by internal combustion (IC) engines

6.4.1 Fuel Cells

Fuel cells are electrochemical devices that combine hydrogen fuel and oxygen from the air to produce electricity, heat and water. Fuel cells operate without combustion, so they are virtually pollution free. However, to operate on digester gas, the gas must be scrubbed of hydrogen sulfide prior to injection into the unit. The fuel cell itself has no moving parts - making it a quiet and reliable source of power. Although initially quite high in cost, continuing R&D are bringing down the cost to levels where they are being used in selected stationery situations. Unit costs of generating capacity by fuel cells are in the range of $1,500 to $2,000 per kW of installed capacity, including gas scrubbing. Hence, for a fuel cell system that would supply 750 kW of electricity, the estimated additional installed cost would be approximately $1.3 million. Energy conversion is in the 26% to 30% range. Grants may be available from the California Energy Commission (CEC).

The City of Portland selected UTC Fuel Cells to install one of its 200-kilowatt fuel cells for converting digester gas, generated by the wastewater treatment facility, into usable heat and electricity for the facility. By using waste gas that might otherwise be flared, the project makes use of a free source of fuel.

6.4.2 Micro-Turbines

Micro-Turbines are similar to small jet engines that burn either natural gas or biogas (digester gas). Specially designed micro-turbines that burn biogas are provided with emission controls that result in emissions with significantly less NOx and other air pollutants than those from reciprocating engine generator sets. Capstone MicroTurbine™ of Chatsworth, CA manufactures a standard 30kW unit, named C30. This unit is designed to burn biogas. Multiple units can be coupled in parallel for a larger output system. Unit costs of installed generating capacity are in the range of $500 to $700 per kW of installed capacity. Hence, for a micro-turbine system that would supply 750 kW of electricity, the estimated additional installed cost would be approximately $0.5 million. Energy conversion is similar to fuel cells and is in the 26% to 30% range. Heat recovery systems can be attached to the units. Grants may be available from the California Energy Commission (CEC).

The San Elijo JPA installed a 3-unit (30C) system at its 3 mgd Water Reclamation Facility. The system burns digester gas, generates 80kW of electricity and recovers waste heat to heat the digesters. The system began operation in 2002 and avoids purchasing equivalent amount of electricity from the utility.

6.4.3 IC Engines

Internal combustion engines, driving electrical generators, can run on natural gas, digester gas or a blend of the two. In order to be permitted by the AQMD, engines must be of the "clean burn" type, which generally come in the size of 1 mW and larger. Since there is considerable heat lost from an IC engine, the heat is usually recovered for purposes as heating digester sludge and/or buildings. This type of system is called Cogeneration, of CoGen. Unit costs of installed generating capacity for CoGen systems are in the range of $1,000 to $1,200 per kW of installed capacity in the 1 mW size. This cost range includes heat recovery (which has a value) and emissions controls. Energy conversion is in the 35% to 40% range.

The determination of the applicability of CoGen to the Water Reuse Project would include analysis of digester gas production (amounts and BTU value seasonally) and residual amounts remaining after consuming major portions in the existing boilers which heat the digester sludge.

CoGen is used at several WWTP, including City of Stockton, Union Sanitary District, EBMUD and several others. Generally, it has been implemented at larger plants.

6.5 Cogen Power Supply from Valero

Valero owns and operates a cogeneration project. Consideration could be given to exploring the possibility of the refinery to supply power to the Water Reuse Project. The basic concept would be to utilize the refinery cogeneration power generation capacity to power the Reuse Plant over the PG&E power lines. (In the initial discussions with PG&E, they were asked to comment on the viability of this concept.) CDM (TGC) contacted Valero (A.Ng) on 19 January 2005, and CDM was advised that Valero consumes all of the power produced by its on-site cogeneration system. No discussions were had regarding future cogen projects planned by Valero.

6.6 Summary of Alternative Power Sources

Owing to reliability considerations and high initial installed cost, alternative power supplies should be considered only as supplemental sources, which, if found economically feasible, would reduce power supplied from PG&E. Wind turbines are a possibility, although visual impacts are of concern. Solar PV systems take up too much land. Possibly a small system could be mounted on building roofs.

The most promising supplemental source would come from those systems that run on the renewable fuel, digester gas (or biogas). The amount of digester gas available will dictate the size of system that could be implemented.

To determine the feasibility of providing an alternative energy supply system to supplement to power demands of the Water Reuse Project would require process analyses of the WWTP anaerobic digestion process, development of estimated cost (capital and O&M) of alternative systems, discussions with PG&E and the CPUC Alternative Energy Program regarding amounts of rebates available and a determination of how these systems could be tied into the electrical supply at the WWTP. Using all of the above information, an economic payback analysis would then be performed.

Performance of the above engineering and economic analyses is beyond the scope of work in CDM contract with the City for development of the Water Reuse Project.